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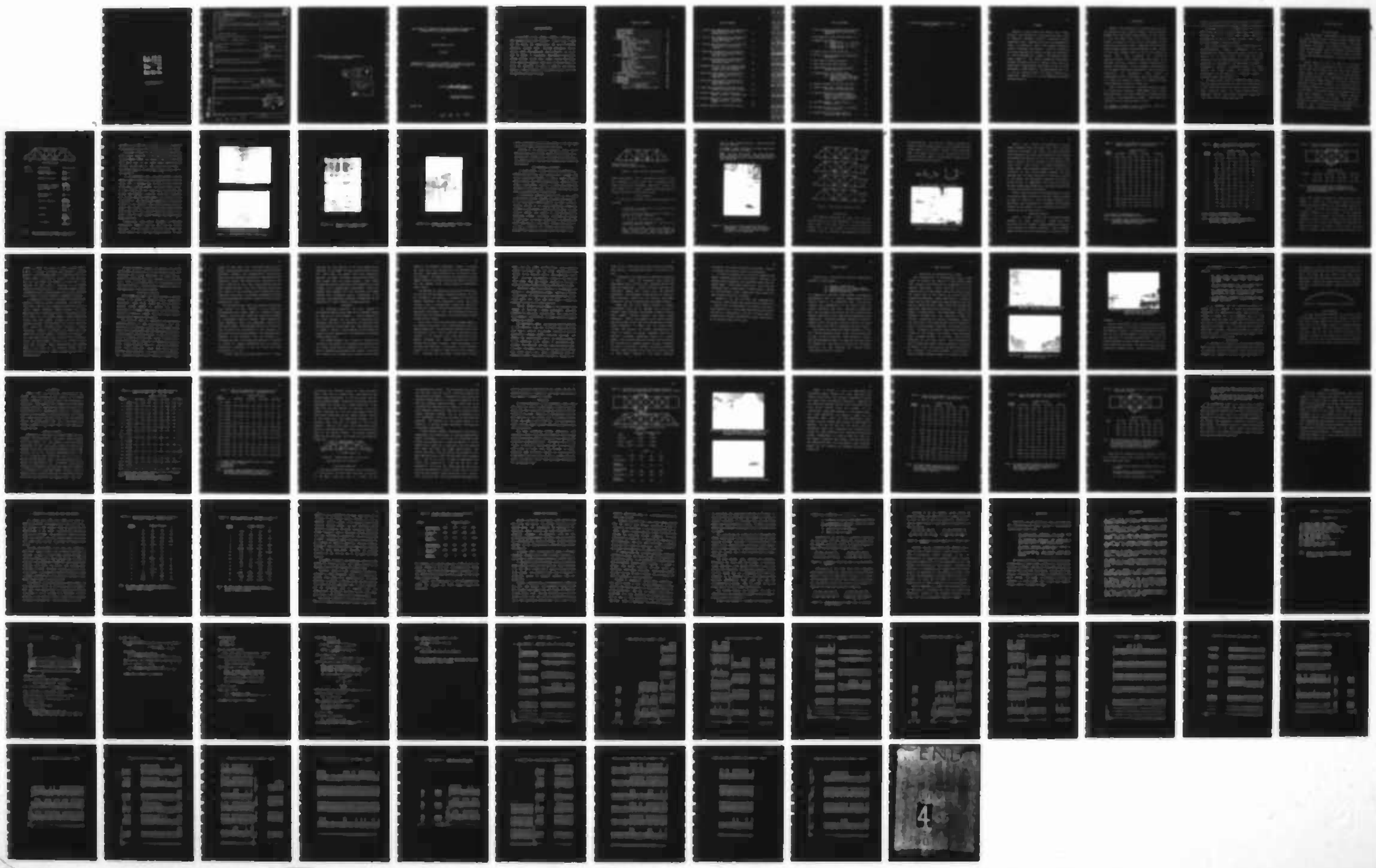

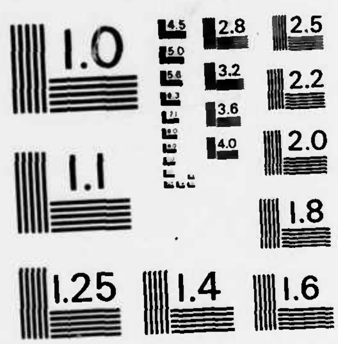

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LOAD TESTS OF A THROUGH TRUSS BRIDGE BEFORE AND AFTER THE INSTALLATION OF AN ARCH-SUPERPOSITION SCHEME

by

Richard Eugene Haller

A Thesis

Presented to the Faculty of Bucknell University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

Approved: ____________________
Advisor

____
Department Chairperson

August 1983

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ABSTRACT

Because so many pinned and riveted truss bridges currently are classified as structurally deficient, a need to develop an economical, practical alternative to bridge replacement exists. Rehabilitation by an arch-superposition scheme can effectively reduce truss member stresses and increase the bridge load capacity. This was verified by load tests conducted both before and after the installation of the arch-superposition scheme to Northumberland County bridge number 50. The results showed that truss member stresses and deflections can be reduced at least 50% by the scheme which is approximately one-seventh the cost of bridge replacement. Bridge number 50 has been upgraded from a six to a 20 ton inventory rating. The theory, economics, and construction feasibility of the arch-superposition scheme have been verified.

1. INTRODUCTION

Numerous articles have been published recently on the dire condition of the country's bridges. There are 568,000 federally and state owned roadway bridges in this country, of which 72% where built prior to 1935. Also, 373,000 bridges are owned by county and local governments, 92% of which were built prior to 1935. Based upon results from the federal bridge inspection program completed in 1981, 106,923 of the state bridges (both on and off the federal aid system) have been classified as structurally deficient--unable to carry today's heavy truck loads. It is estimated that $41.1 billion is needed to rehabilitate or replace these bridges. Clearly, spending this much money is, at present, politically unacceptable. Thus, alternative structurally and economically viable methods of rehabilitation must be developed. Due to continuing decay, the number of deficient bridges will increase each year unless appropriate measures are taken [1].

Many of the structurally deficient bridges are pinned or riveted truss bridges, a common design prior to 1935. Where applicable, truss type bridges can be rehabilitated by utilizing an arch-superposition scheme. This scheme would increase the load carrying capacity of the

[1] Numbers in brackets refer to references listed in the bibliography at the end of the thesis.

bridge to at least 20 tons with minimal traffic interruption since the bridge need not be closed to complete the work.

The bridge rehabilitation is completed by superimposing steel arches upon each truss and adding floor beams between existing ones [2]. In this manner, bridge loads can be increased and the threat of sudden failure, a major concern for truss bridges, is greatly reduced.

This thesis addresses the actual effectiveness of the arch-superposition scheme. Measurements of truss deflections and member stresses were taken during the load testing of an actual bridge both before and after installation of the arch-superposition scheme. Further, a comparison of experimental and theoretical stresses is presented to determine differences between the design (ideal) and actual trusses, the reasons for these differences and the ability of a computer program to accurately model the structure.

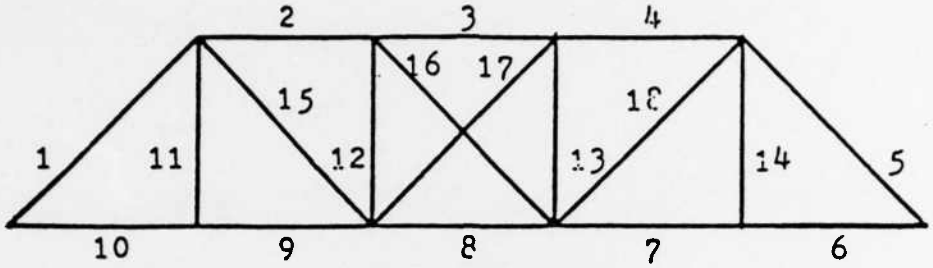
The body of this thesis is broken into three sections. The first section describes the initial test conducted prior to the arch installation, the second section describes the test conducted after the bridge rehabilitation was completed, and the final section presents a comparison of test results from the first two sections.

2. INITIAL FIELD TEST

The Test Bridge

The test bridge, Northumberland County bridge number 50, spans Roaring Creek, located in central Pennsylvania between Northumberland and Columbia Counties. The region surrounding the bridge is principally occupied by camping areas and summer homes. Due to the bridge's five-ton weight limit, large campers, fuel oil delivery trucks, and snow removal and firefighting equipment are forced to detour the bridge.

Bridge number 50, built in 1895, is a five-panel through truss with a total length of 72'-11" and height of 16'-0". The deck is made up of two by four inch (nominal) timber planks (on edge) supported by six by twelve inch (full) timber stringers. The floor beams, which are twelve inch deep Wide Flange sections, are rated at eight-tons operating and six-tons inventory. An operating rating is the maximum allowable live load that can be safely supported by the bridge. An inventory rating is the maximum live load that can be applied to the bridge an infinite number of times without affecting its strength. The floor beams control the allowable bridge load capacity (see Appendix 1). Figures 1 and 2 contain sketches and photographs of the bridge and member cross sections. Both of the bridge



Looking Upstream

Member	Description	Cross Section
6,7,9,10	Lower Chord Tension Eye Bars	½" x 2"
8	Same as 6,7,9,10	3/4" x 2"
1-5	Top Chord - Composed of 2 Channels and Cover Plate	10" 5"
11,14	First and last Verticals	15/16" sq.
12,13	Interior Verticals	10" 4"
15,18	Diagonals	½" x 2"
16,17	Counters	1" sq.
19	Floor Beams	5½" 12"

Figure 1: Truss Dimensions and Member Cross Sections. Northumberland County Bridge Number 50.

abutements are constructed of masonry with the near abutment (Northumberland County side) also having a concrete jacket.

The damage to several of the truss members affects their load carrying capacities. On the upstream truss the first vertical member (number 14 in Figure 1) made up of two 15/16 inch square bars is bent about twenty degrees from the vertical, two feet from the lower chord pin (see Figure 3). This damage appears to be from debris impact during a period of high water.

On the downstream truss, the second interior vertical member (number 12) made up of two channel sections joined by lacing bars is damaged from an auto collision with the bridge railing near the member. The inside channel section is bent outward and the lacing bars are buckled. The inside, near diagonal, a one-half by two inch rectangular bar is very loose and would appear not to carry any load (see Figure 3). The near-end post composed of a cover plate riveted to two channels, has rotated outwards from auto impact. This rotation in turn bent the first outside eyebar of the lower chord. The outside, lower chord eyebar of the center panel is bent upward possibly from debris impact during high water.

Additionally, four timber stringers are severely deteriorated although three of the four are outside stringers. The top flange of all four floor beams have section losses of up to 50% due to heavy corrosion. At some

Looking Downstream

Looking from Northumberland to Columbia Counties.
Upstream truss to the right.

Figure 2: Northumberland County Bridge Number 50

Figure 3(a): Damaged truss members, member number 14 - upstream truss

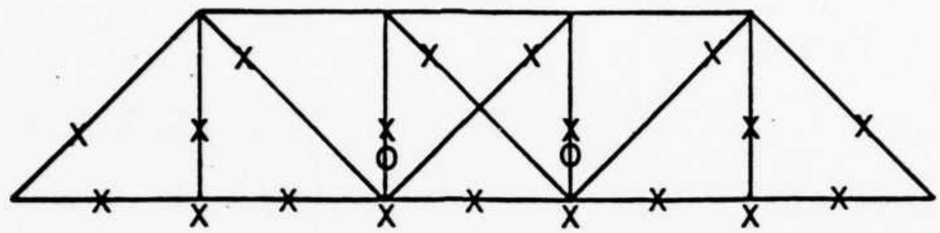
Figure 3(b): Damaged truss members, member number 18 - downstream truss time a one-quarter inch plate was welded to the top flange of the floor beams providing additional area.

Because of the eight-ton operating rating, the low underclearance allowing easy access, and the relatively low average daily traffic count, the bridge is an ideal choice for application of both the arch-superposition scheme and load testing.

Test Equipment and Theory

To record member strains, electrical resistance strain gauges were used along with a digital strain recorder. The strain gauges were attached to a select number of members whose surfaces were first sandblasted and ground to form a smooth, clean surface. Pigtails, or short lead wires, were soldered to each gauge prior to covering the gauge and tabs with a waterproofing solution and putty.

A preliminary computer analysis of one of the trusses indicated the vertical, diagonal, and lower chord members to be the most highly stressed members. In order to obtain a complete picture of the stress distribution, gauges were attached to these members along with the end posts, interior vertical members, and counters on both the upstream and downstream trusses. In addition, two gauges were attached to each U-bolt, one on each side of the U, to determine the exact loads applied to each truss. Knowing these loads, one is able to determine any possible load distribution to adjacent floor beams by the deck system (see Figure 4 for a



X=strain gauge location
O=deflection dial gauge location

Figure 4: Strain and dial gauge locations

gauge layout sketch). Since the members selected for study are subjected to axial loads only, the gauges were not always placed along the member centerline. Instead, they were placed at a location requiring the least amount of grinding and hence the least section loss.

Test Procedure

To load test the bridge, the following procedure was followed:

(1) Connect the lead wires to the gauges and the digital strain recorder

(2) Set up the dial gauges as shown in Figure 5 at each of the interior verticals

(3) Calibrate and balance the recorder

(4) Record the zero readings for both the strain gauges and deflection dials

(5a) Move the test vehicle (H loading) to positions one through four, placing the truck rear wheels directly over each floor beam and straddling the roadway centerline (see Figure 6)

(5b) At each loading position, record the gauge and dial readings

(6) Repeat steps 3 through 5 at least twice more to verify reading consistency.

NOTE: For the first trial only, mark the exact wheel positions in order to locate the truck in the same position for all trials conducted and the final load test.

Figure 5: Set up of the dial gauges for deflection measurement. A thin wire from the dial to the weight in the water measures movement relative to the stream bed.

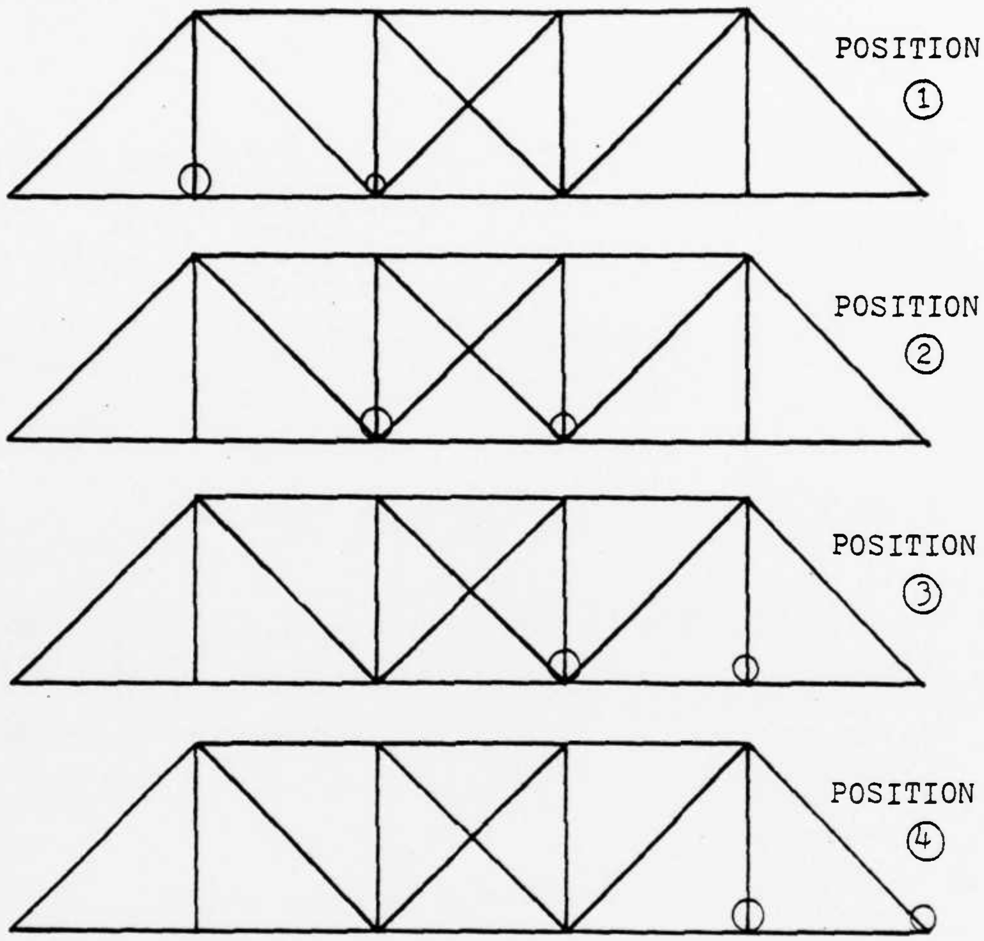
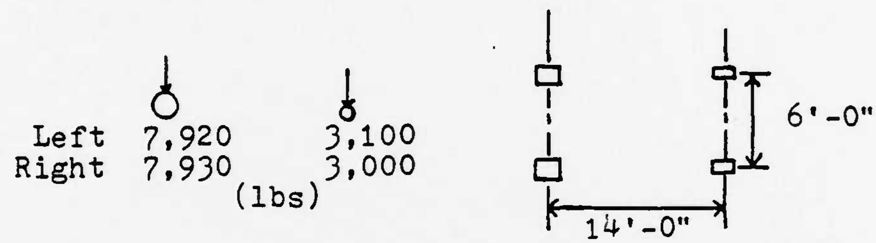


Figure 6: Test vehicle loading positions

Test Results

The bridge was load tested on a sunny, warm (72 F) day. The quasi-static live load was applied by a Northumberland County gravel truck (see Figure 7). For load cases one through four, the rear wheels of the truck were placed directly over the floor beam and symmetrically about the bridge centerline. Of a total of 76 gauges, readings were obtained from 54 gauges located on the trusses and eight gauges located on the U-bolts. Because of the steep creek bank, dial gauges were set up at only three of the four interior vertical members. Of these three gauges, one was defective giving erratic results.



Truck wheel weights and axle spacing

Two-axle gravel truck used to apply the live load

Figure 7: Test vehicle and wheel weights

Tables 1 and 2 display the experimental, live load member stresses in kips per square inch (ksi) for loading positions one through four. These values were derived by first subtracting the initial zero reading from the readings for positions one through four for all three trials. Gauge strain readings are the average of the readings for the three trials. Since strain is directly proportional to stress, these strain readings multiplied by the modulus of elasticity (E) give average member stresses. For steel produced prior to 1905, the modulus of elasticity as given by the Manual for Maintenance Inspection of Bridges is 29,000 ksi [3]. A comparison of the three trial readings for each member gauge showed good consistency between each of the trials for most of the gauges.

Table 3 lists the truss live load deflections in inches for the dial gauges located on the interior vertical members as shown in Figure 4. Like the member stresses, truss deflections were determined by averaging the difference between the zero reading and the readings for positions one through four for all three trials.

Analytical Results

Tables 1 through 3 also list the predicted member stresses and truss deflections. These values were calculated with the computer package PFENNIGS developed by Vince Mehringer of Bucknell University. Within PFENNIGS, member stresses and truss deflections are calculated by the

Table 1: Initial test experimental and theoretical truss member stresses (ksi) - upstream truss

Member Number	Loading Position 1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1	-1.54	-1.57	-1.22	-1.16	-.84	-.73	-.35	-.31
5	-.38	-.54	-1.16	-.97	-1.86	-1.39	-1.16	-1.22
6	-.75	-.29	-.26	.60	.99	1.72	1.74	.53
7	1.60	-.29	2.96	.60	2.99	1.72	-.67	.53
8	.73	1.14	1.91	2.27	1.65	1.83	.55	.50
9	2.20	2.27	.64	1.06	-.61	.07	-.96	-.38
10	1.94	2.27	.55	1.06	-.38	.07	-.64	-.38
11	4.55	4.50	.00	.00	.00	.00	.00	.00
12	.44	.46	.29	.29	-.78	-.66	-.29	-.26
13	-.52	-.48	-.44	-.36	.58	.61	.20	.27
14	.00	.00	.00	.00	.78	1.76	2.81	4.50
15	.00	.17	4.32	4.03	2.93	2.55	1.28	1.07
16	-2.61	-1.85	-1.74	-1.13	3.19	2.67	1.36	1.07
17	1.65	1.94	1.45	1.47	-1.83	-2.43	-1.16	-1.07
18	.38	1.89	.15	3.39	1.02	2.78	1.42	-1.07

Act = Actual or experimental value
Pred = Predicted or theoretical value

Note: For member number designations see Figure 1. Predicted values based upon computer run for pin-60% pin support conditions.

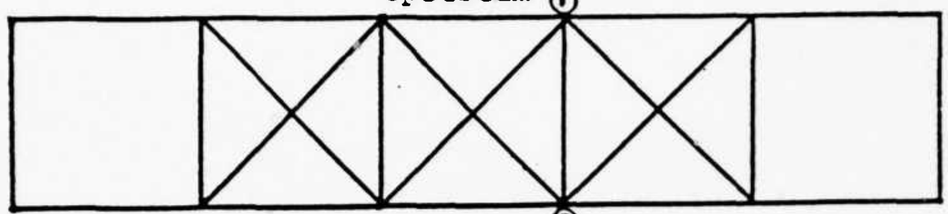
Table 2: Initial test experimental and theoretical truss member stresses (ksi) - downstream truss

Member Number	Loading Position 1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1	-.75	-1.57	-.58	-1.15	-.44	-.73	-.15	-.31
5	-.49	-.54	-.78	-.96	-1.10	-1.38	-.99	-1.22
6	-.70	.02	-.12	.92	.87	2.04	1.42	2.11
7	1.80	.02	3.48	.92	3.22	2.04	-.81	2.11
8	.49	.99	1.77	2.26	2.61	1.61	-.49	.45
9	.35	2.58	-.09	1.39	.20	.42	.64	-.17
10	3.05	2.58	2.09	1.39	1.10	.42	.12	-.17
11	3.42	4.50	.00	.00	-.06	.00	.00	.00
12	4.06	.10	.58	.08	-1.07	-1.13	-2.70	-.48
13	-.38	-.83	-.29	-.60	.15	.15	.15	.06
14	.15	.00	.20	.00	1.54	1.70	4.50	4.51
15	.00	.14	.00	4.01	3.45	2.55	1.10	1.07
16	.09	-.38	-.20	-.24	2.41	4.56	.96	1.92
17	1.74	3.37	1.25	2.41	-.87	-.51	-.58	-.23
18	-.73	1.88	-.09	3.36	.75	2.80	1.13	-1.07

Act = Actual or Experimental Value
Pred = Predicted or Theoretical Value

Note: For member number designations see Figure 1. Predicted values based upon computer run for pin-50% pin support conditions.

Table 3: Initial test experimental and theoretical truss vertical deflections (inches)



	Loading Position							
	1		2		3		4	
Gauge	Act	Pred	Act	Pred	Act	Pred	Act	Pred
1	.030	.043	.086	.090	.100	.099	.037	.035
2	.032	.038	.200	.089	.227	.113	.138	.042

Note: Upstream truss deflections computed with pin-60% pinned support condition, downstream truss with pin-50% pinned support condition. Positive deflection downwards.

plane frame program developed by Weaver and Gere. This program is based upon the stiffness method of analysis assuming structure linearity and elastic member properties.

Initial data inputs were based upon the results of a thorough inspection of the bridge. At both supports movement in the horizontal and vertical directions was prevented to model the pin supports. The cross sectional area of all the lower chord members was reduced by ten percent to account for corrosion of the steel. The pins along the lower chord were fixed against rotation in an attempt to model the action of these corroded joints. Finally, to prevent the program from assigning a bending moment to a pin-connected member or a compressive force to a slender member, the moment of inertia or cross sectional area for the member was reduced to nearly zero. This reduction was done after an inspection of the member stresses calculated initially. Any slender member assigned a compressive load or bending moment had its cross sectional area or moment of inertia reduced in subsequent analyses.

A comparison of the actual and predicted stress results revealed a need for a better correlation. By modeling the truss twice, once with pin supports and once with a pin and roller support, predicted member stresses and truss deflections can be determined for the range between 0% and 100% pinned for the second support. Another comparison between the actual member stresses and the range of predicted stresses indicates many of the actual values now fall within the predicted range achieving a better correlation. However, the effect of the damaged and loose members has not been modeled. In addition, the behavior of the lower chord pinned joints has only been estimated by fixing the joints against rotation in an attempt to model their high friction. These shortcomings in the computer model of the truss are the principal reasons for the deviations of some actual member stresses from the range of predicted values.

As with the member stresses, the upstream truss actual deflections correspond well with the predicted values. The downstream truss actual deflections are higher than the predicted deflections for load positions two, three and four. This is principally due to the looseness of the joints which allows a certain amount of deflection prior to load application to the truss members.

Other Tests Conducted

Many bridges have been load tested over time by various methods in an effort to determine actual stresses and deflections. These actual values can then be used to evaluate current methods of analysis and design, particularily the AASHTO specifications for the design and rating of all types of bridges. With this in mind, several through truss bridges have been tested.

One series of tests was conducted by the Engineering Research Institute at Iowa State University [5]. The objective of the tests was to compare the 1975 AASHTO bridge design and rating procedure with the observed field behavior of pin-connected, high truss bridges. A total of four spans from two separate bridges were tested. One span from a Parker truss bridge was tested to its ultimate load capacity while a second and third span and a span from a modified Pratt truss bridge were tested under service loads. Recorded member forces and truss deflections were compared with theoretical forces computed assuming all members were joined by ideal pins. For the ultimate load test, most strain readings agreed closely with the theoretical forces calculated. However, the load-deflection curves for the lower chord pins differed considerably in magnitude from the theoretical curves. This behavior was attributed to "the frozen condition of the truss joints resulting from the rusted members and pins " [5, p. 26]. From the ultimate load test results it was concluded that "although the actual conditions in the joints are unknown, considering the truss to be pin-connected does provide a realistic method of truss analysis" [5, p. 26].

For the service load tests, the procedure followed was very similar to that followed in the testing of Northumberland County bridge number 50. Theoretical member influence lines and truss deflection values were developed be a determinate method of analysis. When compared with these theoretical values the experimental results were consistently lower. This difference was due to the partial deck continuity (not accounted for in the analysis), the condition of the joints and problems in the instrumentation. Therefore, it was concluded that "the analysis of a pin-connected truss, even though the condition of the pins is unknown, as a simple determinate truss will provide a conservative indication of the bar forces and truss deflections" [5, p. 32].

A second series of tests was conducted by C. P. Heins involving the load testing of six truss bridges and the evaluation of the AASHTO stringer load distribution factor [6]. Both static and dynamic tests were conducted to determine the effect of wooden nailing strips attached to each stringer upon the bridge capacity. Total member stresses were found by adding the field determined live load stresses to the computed dead load stresses. These total stresses when compared with the analytical total member stresses were consistently smaller in magnitude. It was concluded that determinate analytical methods used are too conservative in their estimate of bridge capacities.

Discussion

As previously mentioned, the truss supports of Northumberland County bridge number 50 act somewhere between an ideal pin and- roller support. A careful comparison of the actual stresses and the range of predicted values indicates a best fit at 50% pinned for the downstream truss and 60% pinned for the upstream truss. The predicted values in Tables 1 through 3 are determined with pin-50% pin or pin-60% pin end supports for the downstream and upstream trusses respectively.

The member stresses for a truss with pin-50% pin or pin-60% pin end supports are calculated by multiplying both the pin-pin and pin-roller stresses by 0.5 and 0.5 or 0.6 and 0.4 respectively and adding them together. The 50 and 60% figures are an average of the percent values for each member of the upstream or downstream trusses. (An average for each of the four load positions was determined first. The final truss support conditions are an average of these four averages.) The member percent value represents the end support conditions at which the predicted stress (calculated with the computer model) exactly equals the experimental member stress.

The upstream and downstream trusses acted differently. Not only were the end support conditions different, but also the stresses in the counters. To improve the correlation between theoretical and experimental values, the cross sectional area of the downstream counters was reduced to one-tenth of a square inch while the upstream counters were maintained at one square inch. This correction in area was required since the compression in the slender members varied between the two trusses.

Since there is only one significantly damaged member in the upstream truss versus five in the downstream truss, the member stresses and truss deflections correlate better with the predicted values. For all four load cases most of the stresses in the upstream truss lower chord and vertical members correspond well. However, some deviation in the diagonals and counters is present. There would appear to be two additional causes for this deviation. First, the panel length of 14'7" is slightly longer than the truck length of 14'0". Therefore, the floor beams located near the truck front axle are loaded "eccentrically". This eccentric loading is evident from the U-bolt readings which read tension on one side and zero or even compression on the other. Also this was sometimes evident on the rear axle floor beams. From the "frozen" condition of the pins, a load distribution different from an ideal pin then occurs.

The second cause for the deviation in the diagonals and counters is the differences in member tightness or length. A tight member will begin carrying load immediately, while a loose member will carry load only after a sufficient deflection occurs at the lower chord pin.

The one damaged vertical member of the upstream truss (member 14) exhibited a stress considerably lower than predicted for load cases three and four. This lower stress would suggest a redistribution of member forces to adjacent members has occurred.

The experimental member stresses and truss deflections of the downstream truss vary from their predicted values more than do those of the upstream truss due to the five significantly damaged members previously mentioned. For example, the loose diagonal, one of two composing member 8 of the downstream truss, results in considerably lower stresses than predicted for load cases one through three. In general, the members close to the load application points had stress values that varied considerably from the predicted stresses indicating a transfer of loading to the truss by the U-bolts and pins other than that predicted. This variation is particularly true for load cases one and four.

A comparison of the upstream and downstream deflections in Table 3 shows the downstream truss deflecting approximately twice as much as the upstream truss for load cases two through four. Further, the actual upstream truss deflections compare very closely with the predicted deflections. The downstream deflections do not. Similarly, good correlation existed for the upstream truss member stresses but not for the downstream truss member stresses.

Based upon the test results obtained, a typical pin-connected truss can be analyzed with the aide of a computer model utilizing the stiffness method of analysis which assumes structure linearity and elastic member properties. However, the accuracy of the analysis depends upon the accuracy of the input data obtained from a thorough bridge inspection. To ensure that the maximum member stresses are obtained, the counter cross-sectional area should be reduced as required (depending upon the load position) to prevent compressive loading of a slender member. Also the truss should be analyzed with both pin-pin and pin-roller end supports despite the type of supports that exist in the field. Although the majority of members are most highly stressed in the pin-roller condition, some are most highly stressed in the pin-pin condition, principally the interior verticals and counters. Finally, all possible load positions should be investigated.

It should be stressed that the analysis assumes uniform member properties along the entire length. Therefore, the minimum section properties are considered uniform along the member length. This minimum section will often occur near connections and in areas subject to roadway splash. Because of limited access to many connections, their condition is often the most difficult to evaluate.

Sanders et al. [5] and Heins [6] determined that analyzing a truss bridge as pin connected and determinate is a realistic way of establishing the ultimate and service load capacities of a truss bridge provided the partial deck continuity is accounted for. The test of bridge number 50 verified this conclusion provided that careful consideration of the analysis input data be given.

Chapter Summary

There are four major kinds of structural imperfections possible in truss bridges:

(1) Geometric imperfections
(2) Imperfections in the loading system
(3) Imperfections in the support conditions
(4) Material imperfections

An inspection of the test bridge revealed that geometric imperfections exist in numerous damaged members with the majority being in the downstream truss. The test data revealed that imperfections in the loading system and support conditions also exist. The best fit theoretical data was calculated with pin-60% pin and pin-50% pin end support conditions for the upstream and downstream trusses respectively. The U-bolt readings revealed an "eccentric" loading of the floor beams since one side of the U-bolt carried most of the front and sometimes rear wheel loading.

The initial test proves that simple analyses of a truss will provide reasonable member stress values provided care is taken to model the existing truss conditions. Further, when maximum member stresses are required, the truss should be analyzed with both pin-pin and pin-roller end support conditions, all load positions, and reduced counter cross-sectional areas as required.

3. FINAL FIELD TEST

Description of Arch-Superposition Scheme

The rehabilitation of Northumberland County bridge number 50 required the addition of steel arches, hanger rods, stringers, floor beams and the replacement of the four existing floor beams. A steel arch, made up of ten line segments having the same channel section but varying length, was placed just to the outside and inside of both the upstream and downstream trusses (see Figure 8). Additional hanger rods and floor beams were placed between the existing vertical members and floor beams, halving the stringer span length and improving the arch efficiency by increasing the number of loading points. The reduced span length of the stringers increased their load capacities to twenty tons (H20-44), eliminating the need for replacement. However, the two by four inch timber decking was not sufficient to carry an H20-44 loading. There were two possible options: first, to replace the existing timber deck (requiring the closing of the bridge), and second, to place additional steel stringers between the existing wood stringers shortening the deck span length. Since the addition of steel stringers was lower in cost and required no disruption of traffic, it was the alternative selected (see Figure 9). For the design of the steel stringers, care was taken to match the member stiffness closely to the timber stringer

Figure 8: Downstream truss with the truss-arch scheme installed. New members have primer paint.

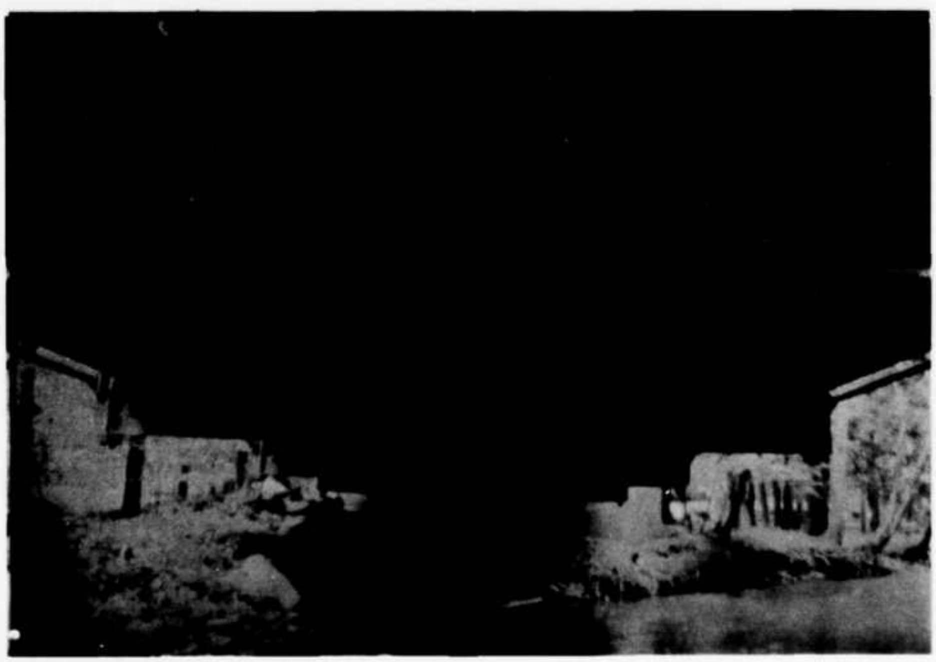


Figure 9(a): Strengthening of the timber deck. Existing deck system.

Figure 9(b): Strengthening of the timber deck. Additional steel stringers added to shorten the deck span length.

stiffness.

With up to 50% section loss on the top flange and only an eight-ton capacity, all four of the existing floor beams were replaced. Finally, to withstand the horizontal thrust from the arch, the far abutment (Columbia County) required the addition of a nine-inch concrete jacket or skirt. As already mentioned, the near abutment was jacketed previously after a period of high water. For details on the design and construction schedule of this project see reference number 9.

The advantages of the arch-superposition scheme are as follows.

(1) The bridge load carrying capacity can be increased to twenty tons (H20-44) by the addition of steel arches, hanger rods, floor beams and stringers.

(2) The rehabilitation will not require complete closure of the bridge, only short delays.

(3) Considerable cost savings can be achieved by eliminating the need to replace the bridge, particularily at today's design standards.

(4) Redundancy is added to the bridge which will reduce the likelihood of collapse should a truss member or pin fail.

It is predicted that the trusses and arches of the bridge will act together to carry the applied loadings effectively. The percent of the total load carried by the trusses or arches will vary depending upon the load location. When the truck rear and front wheels are located near one of the added floor beams, most of the load will be carried by the arches. For a load location near "existing" floor beams, a greater load sharing between the trusses and arches will occur. In any load position the trusses provide lateral stability to the arches.

Test Theory

To determine the validity of the truss-arch computer model used for the arch design, strain gauges were placed at the midlength of the first, second, sixth, ninth and tenth straight line segments on all four arches (see Figure 10).

Two gauges were placed at each midlength, approximately four and one-half inches above and below the channel's centroidal axis. One gauge was also placed on each hanger rod to determine the load distribution from the deck to the trusses and arches. Finally, the gauges from the initial load test were used to measure the percent reduction in truss member stresses.

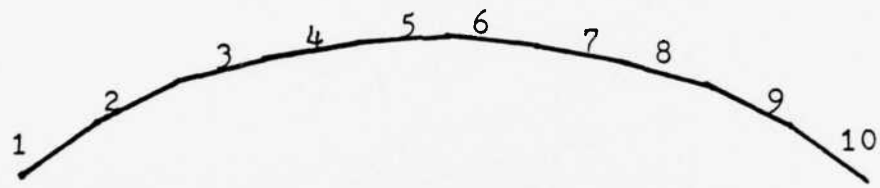


Figure 10: Number Designations for Arch Sections

Test Procedure

The same test procedure employed for the initial load test was used for the final load test. However, since there were now 130 strain gauges on the bridge, each trial was run twice due to the limited number of instrument channels available. The same four load positions from the initial test were used and care was taken to insure exact load placement for each of the three trial and two test runs. Because of the two test runs, deflection data was collected for a total of six trials, three trials from each test run.

Test Results

Tables 4 and 5 list the actual results from the arch gauges. These stresses are the product of the member strains (average of the three trials) and the modulus of elasticity. They represent the combined axial and bending stresses at points away from the channel centroidal axis. The existence of the bending moment results in a difference between the two stresses at each cross section. The highest arch member stresses monitored occur in the end sections nearest the loaded hanger rods. Of a total number of 40 gauges attached to both arches, only 27 were operational during the test.

Analytical Results

Examination of the hanger rod stresses showed the truck loadings to be distributed to five floor beams and hanger rods (see Figure 11). The percent of the total weight distributed to each floor beam varied for each test The percent of the total truck weight distributed to each hanger rod was 9, 44, 21, 22, and 4% for load position one, 7, 66, 12, 11, and 4% for load position two, and 11, 54, 19, 12, and 4% for load position three. Only three hanger rods are loaded for load position four. The percent values are an average of the upstream and downstream hanger rod readings. In determining the predicted member stresses listed in Tables 4 and 5, the load distribution as determined from the field test was entered into the computer model. This load

Table 4: Final test experimental and theoretical arch section stresses (ksi) - upstream truss

Arch Section	c	1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1-I	4.000	-.44	-.87	-.29	-.49	-.38	-.62	-.20	-.69
1-I	-4.125	-.90	-2.01	-1.16	-.58	-.84	-.44	-.46	-.73
1-O	4.250	-.58	-.83	-.32	-.49	-.35	-.62	-.15	-.69
1-O	-3.875	-.87	-1.97	-1.25	-.58	-.96	-.44	-.58	-.73
2-I	4.062	-.75	-3.53	-.06	.22	-.06	.50	-.12	.55
2-I	-3.312	-.35	.47	-1.22	-1.16	-.99	-1.38	-.49	-1.75
2-O	4.437	-1.02	-3.74	-.15	.29	-.09	.60	-.09	.67
2-O	-4.312	-.26	1.02	-1.65	-1.35	-1.48	-1.64	-.78	-2.07
6-I	4.250	-.52	-2.10	-.99	-1.65	-.73	-.74	.26	2.13
6-I	-4.187	--	--	--	--	--	--	--	--
6-O	4.125	-.44	-2.06	-.90	-1.61	-.64	-.73	.26	2.05
6-O	-4.062	--	--	--	--	--	--	--	--
9-I	4.000	-.44	.65	-.55	.44	-.44	-.65	-.96	-4.48
9-I	-4.250	--	--	--	--	--	--	--	--
9-O	4.125	-.58	.69	-.70	.47	-.52	-.65	-1.65	-4.58
9-O	-4.063	--	--	--	--	--	--	--	--
10-I	4.125	-.49	-.80	-.49	-.61	-.29	-.61	-.12	-.65
10-I	-4.250	-.75	-.71	-1.16	-.44	-1.28	-.89	-.78	-2.00
10-O	4.125	--	--	--	--	--	--	--	--
10-O	-4.312	-.93	-.71	-1.33	-.43	-1.31	-.89	-.58	-2.01

Loading Position columns: 1, 2, 3, 4.

c = distance from the channel centroidal axis to the gauge
I = inside arch O = outside arch
Note: For member number designations see Figure 1. Predicted values based upon computer run for 40% fixed-40% fixed arch support conditions.

Table 5: Final test experimental and theoretical arch section stresses (ksi) - downstream truss

Arch Section	c	Loading Position 1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1-I	4.187	--	--	--	--	--	--	--	--
1-I	-4.312	-.09	-2.33	-1.62	-.43	-1.62	-.48	-.75	-.66
1-O	4.125	--	--	--	--	--	--	--	--
1-O	-4.250	-.12	-2.32	-1.83	-.43	-1.91	-.49	-1.10	-.66
2-I	4.438	-.52	-3.89	-.32	.44	-.49	.64	-.23	.63
2-I	-4.250	--	--	--	--	--	--	--	--
2-O	4.187	-.29	-3.75	-.32	.38	-.58	.57	-.26	.56
2-O	-4.125	-1.04	.98	-1.04	-1.41	-.70	-1.92	-.26	-2.06
9-I	4.125	.29	.64	.52	.34	-.12	-1.45	-.55	-4.18
9-I	-4.125	-1.10	-2.21	-1.65	-1.46	-1.31	-.54	.15	1.53
9-O	4.312	.32	.70	.52	.38	-.17	-1.46	-.61	-4.31
9-O	-4.250	--	--	--	--	--	--	--	--
10-I	4.187	-.52	-.95	-.55	-.65	-.06	-.60	.03	-.38
10-I	-4.312	-.38	-.61	-.73	-.46	-1.51	-1.48	-.70	-2.44
10-O	4.125	-.41	-.94	-.44	-.65	-.09	-.61	-.03	-.39
10-O	-4.125	-.41	-.62	-.73	-.46	-1.51	-1.47	-.46	-2.40

c = distance from channel centroidal axis to the gauge - (+) above and (-) below.
I = inside arch
O = outside arch

Note: For member number designations see Figure 1. Predicted values based upon a computer run for 45% fixed-45% fixed arch support conditions.

distribution was adjusted slightly to obtain the actual total truck load as weighed while maintaining the same percent weight distribution. As with the initial load test values, predicted member stresses were calculated with both fixed-fixed and pin-pin arch support conditions. It was found that the arch behavior is sensitive to the arch support conditions but that the truss support conditions have little effect upon the arch member stresses. Because of the actual live load compressive stresses in the counters, their area was not reduced in the live load analysis. Finally, new joints were added twelve inches above the lower chord joints to raise the load application point on the vertical members. This change in geometry models the new connections used between the floor beams and hanger rods.

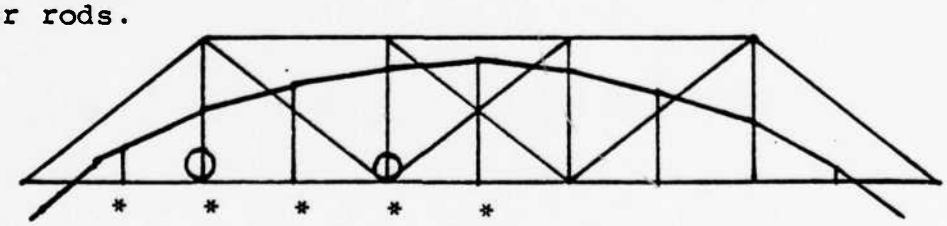


Figure 11: Loaded Hanger Rods - typical for load positions one through four

Other Tests Conducted

A test was conducted by Brungraber, Kim and Yadlosky on a 1:7 scale model of an actual Pratt truss [2]. The testing of the model truss was part of a study of the arch-superposition scheme. Truss deflections were measured for the truss alone and for the truss-arch loaded at the same position and weight. A comparison of these deflections showed an average reduction of deflection readings of 30 to 40% with a maximum reduction of 68%. A final test of the truss-arch was conducted with one section of a truss's lower chord completely removed and again with one section of both trusses completely removed. Due to the presence of the arch, catastrophic failure of the bridge in both cases was prevented. The addition of the arches proved to be effective in reducing bridge deflections and preventing catastrophic bridge failure.

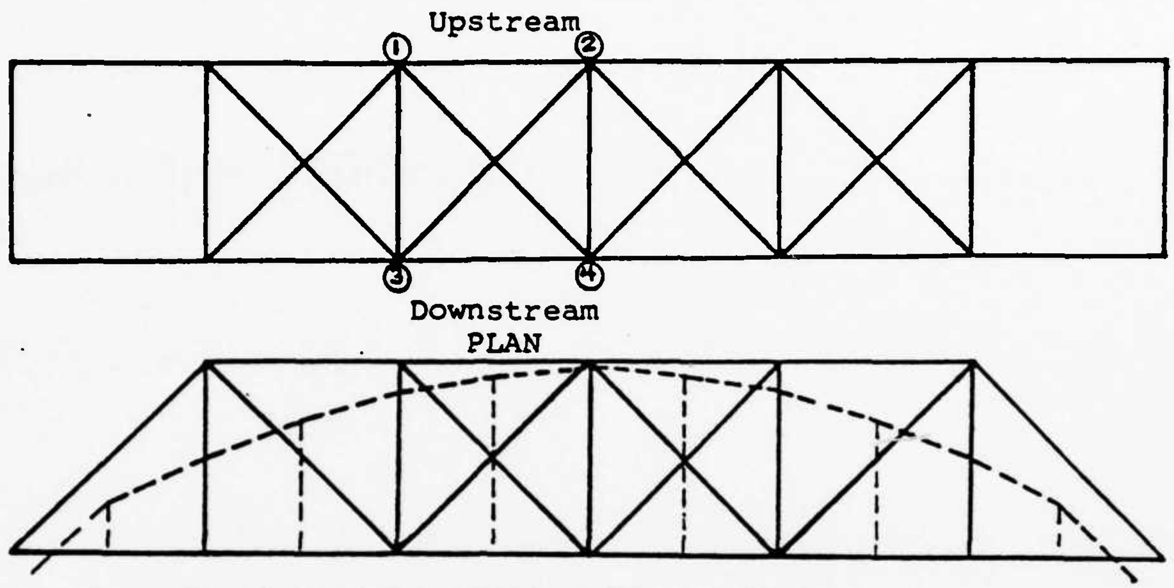
A second bridge located in Coudersport, Potter County, Pennsylvania was rehabilitated by the arch-superposition scheme. The bridge, a 73'-2" long, 15'-0" high Pratt truss bridge, was increased from a three-ton capacity to a 20-ton capacity (see Figure 12). A deflection test was conducted both before and after the bridge rehabilitation. The initial test used a seven-ton vehicle with standard H truck wheel spacings and the final test the same vehicle weighing both 7 and 22 1/2 tons. To permit a comparison of the initial test conducted at 7 tons and the final test conducted at 22 1/2 tons, the deflections from the initial test were multiplied by a factor of 4.598, the ratio of the truck rear axle weights (final test/initial test). Table 6 lists the initial and final extrapolated truss deflections and the truss-arch deflections for the second load and the percent reduction in deflection. The bridge deflection has been reduced an average of 69 percent.

Discussion

A careful examination of the experimental results from the final load test of bridge number 50 shows there is a fairly even load distribution between the inside and outside arches of each truss. Generally, for the upstream truss the end sections of the outside arch are stressed slightly higher than those of the inside arch. For the downstream truss the more highly stressed arch varies depending upon the load position.

The best correlation between experimental and theoretical values occurs with 40% fixed-40% fixed and 45% fixed-45% fixed arch support conditions for the upstream and downstream trusses respectively. The reduced fixity is due in part to the use of neoprene pads under the steel plates (see Figure 13) which allow some movement to occur. Generally, good correlation between the theoretical and experimental values occurs for the first, second, ninth and tenth arch sections for all four load cases, particularly for the upstream truss.

Table 6: Deflection results from the second bridge to have the arch-superposition scheme installed (inches).



---- Steel members added prior to final test
ELEVATION

Test	Truck Axle Weights (lbs) Rear	Front
Initial	7,420	6,340
Final - Load 1	7,215	6,145
Final - Load 2	34,120	10,860

	Gauge 1	3	2	4
Initial	.058	.064	.093	.083
Final-Load 1	.019	.017	.034	.024
Percent Reduction	67	73	63	71
Extrapolated Initial	.267	.294	.428	.382
Final-Load 2	.095	.080	.140	.111
Percent Reduction	64	73	67	71

Figure 12: Looking upstream at a second bridge to have the arch-superposition scheme installed. Coudersport, Potter County, Pennsylvania

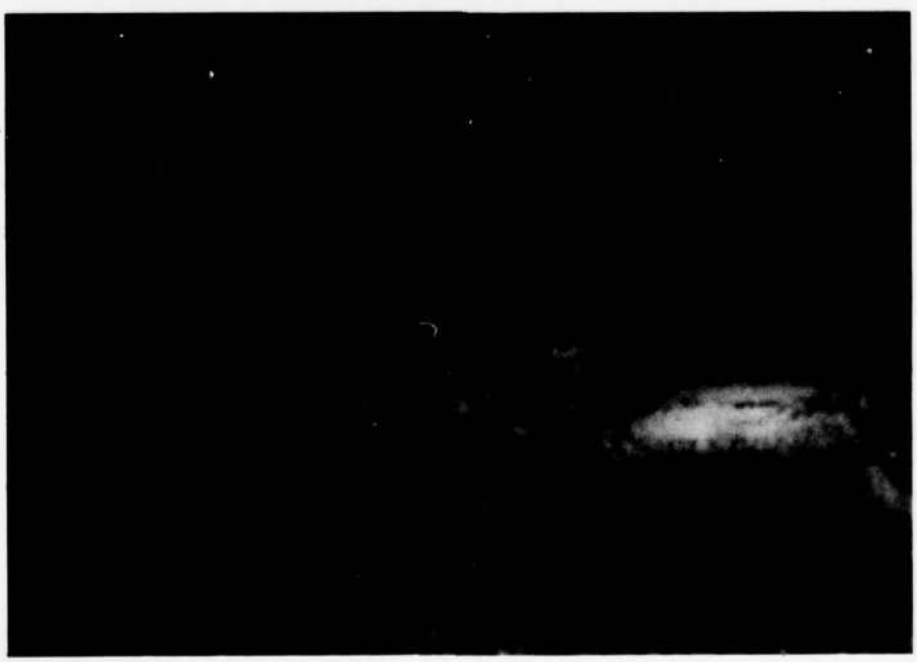


Figure 13: Arch end support - steel plates on neoprene pads.

Tables 7 through 9 list the experimental and theoretical member stresses and truss deflections ones tested in the initial test allowing a comparison of "before and after" truss stresses and deflections. Inspection of the lower chord stresses indicates little likelihood of tied arch action occuring since the stresses are not significantly higher than predicted by the computer model. The truss deflections are generally larger for the downstream truss. Since the load distribution varies between the two trusses, some variation in deflections is expected. As was the case for the initial test, the theoretical values were calculated with pin-60% pin and pin-50% pin truss supports for the upstream and downstream trusses, respectively. In order to model the "best fit" arch support conditions, 40% or 45% of the 100% fixed arch end moments were input into the computer model as applied moments for the upstream and downstream truss analyses, respectively.

Table 7: Final test experimental and theoretical truss member stresses (ksi) - upstream truss

Member Number	Loading Position 1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1	-1.33	.00	-.64	-1.03	-.26	-.54	-.03	.44
5	.03	.15	-.26	-.58	-.99	-.88	-1.07	.38
6	-1.02	-.95	-.17	1.45	1.22	1.19	1.54	.47
7	.90	-.95	2.26	1.45	2.03	1.19	-1.54	.47
8	-.12	.00	1.28	2.30	1.13	1.25	.00	-1.13
9	--	--	--	--	--	--	--	--
10	2.06	1.58	.58	1.87	-.38	-.68	-.64	-1.52
11	1.54	.00	-.17	.00	-.17	.00	-.23	.00
12	.55	-.08	.90	.52	-.23	-.08	.09	1.43
13	.12	1.06	-.20	.26	.93	1.19	.26	-.04
14	.03	.00	.03	.00	.17	.00	.44	.00
15	-1.33	.03	3.65	3.60	2.15	1.80	.61	-1.59
16	-3.57	-2.09	-2.09	-.94	4.90	2.58	2.67	1.86
17	3.60	2.65	2.35	1.64	-4.03	-1.96	-2.32	-1.59
18	-.90	-.42	-.23	2.02	1.02	2.77	1.33	-1.47

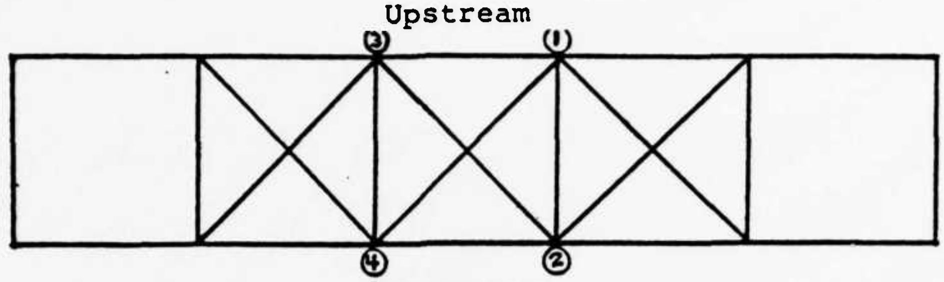
Note: For member number designations see Figure 1. Predicted values based upon computer run for 40% fixed-40% fixed arch and pin-60% pin truss support conditions.

Table 8: Final test experimental and theoretical truss member stresses (ksi) - downstream truss

Member Number	Loading Position 1		2		3		4	
	Act	Pred	Act	Pred	Act	Pred	Act	Pred
1	-1.22	-.05	-.84	-1.18	-.32	-.27	-.12	.43
5	-.15	.15	-.64	-.67	-.99	-.59	-1.51	.30
6	-.49	-.88	.09	.32	.93	1.16	1.02	.38
7	-.93	-.88	.09	.32	1.36	1.16	1.22	.38
8	-.03	.09	1.02	1.85	2.09	.69	-.26	-1.12
9	2.03	1.68	.61	1.02	-.70	-.98	-.75	-1.69
10	2.76	1.68	.90	1.02	-.87	-.98	-1.80	-1.69
11	.93	.00	-.15	.00	.00	.00	-.03	.00
12	.20	.09	.87	1.40	-.75	.49	-.26	1.50
13	-.20	1.10	-.20	-.18	.73	.28	-.12	-.26
14	.09	.00	.15	.00	.35	.00	1.13	.00
15	-1.07	.20	4.38	4.12	2.12	.92	.70	-1.52
16	-1.80	-2.17	-.78	-1.38	6.64	2.53	3.54	2.02
17	5.08	2.82	4.29	1.99	-1.65	-2.29	-.93	-1.83
18	.70	-.40	2.55	2.30	2.47	1.97	-.75	-1.16

Note: For member number designations see Figure 1. Predicted values based upon computer run for 45% fixed-45% fixed arch and pin-50% pin truss support conditions.

Table 9: Final test experimental and theoretical truss-arch deflections (inches)



Gauge	Loading Position 1 Act	1 Pred	2 Act	2 Pred	3 Act	3 Pred	4 Act	4 Pred
1	.001	-.016	.049	.070	.084	.078	-.078	-.037
2	-.001	-.014	.080	.072	.123	.052	.034	-.030
3	.035	.011	.078	.089	.034	.046	-.006	-.051
4	.065	.016	.125	.100	.029	.021	-.020	-.050

Note: Positive deflection downward. Upstream truss deflections computed with 40% fixed arch supports and pin-60% pin truss supports. Downstream truss deflections computed with 45% fixed arch supports and pin-50% pin truss supports.

Based upon the experimental results obtained, to model the truss-arch action most accurately and safely, the computer model should use:

(1) pin-pin arch supports for determining the arch stresses

(2) both pin-pin and pin-roller truss supports to determine the maximum stresses

(3) full counter cross sectional areas

(4) reduced truss member cross sectional areas and moment of inertias as previously described in the initial test .

(5) a load distribution to five hanger rods, the percent of the total H type truck load carried by each rod varying for each load position.

The experimental results also clearly show the H20-44 load capacity of the bridge. The member stresses are well below the maximum allowable for both the trusses and arches. The addition of the arches has increased the degree of certainty of the bridge rating since they add redundancy to the bridge. This redundancy, as shown in the test of the model bridge, has reduced the need to effectively evaluate the capacity of the lower chord pin connections particularily if the arches are designed to carry the H20-44 live load and truss-arch dead load.

Chapter Summary

The load capacity of Northumberland County bridge number 50 was increased from eight to twenty tons by the addition of steel arches, hanger rods, stringers, floor beams and the replacement of the four existing floor beams. This arch-superposition scheme added a redundancy to the bridge as shown by the model bridge test conducted by Brungraber et al. [2] reducing the need for accurate strength estimates of the lower chord connections. The load test experimental results showed that load sharing between the arches and trusses occurs and members in both are not stressed above the maximum allowable. In order to model the truss-arch action accurately, the computer model should use pinned arch supports and a truck load distributed over five hanger rods on each truss.

4. COMPARISON OF INITIAL AND FINAL TEST RESULTS

By testing the bridge both before and after the installation of the arch-superposition scheme, not only can the accuracy of the computer model be checked but the effectiveness of the scheme in carrying and distributing loads can be evaluated. The experimental results from the final test revealed that load sharing between the trusses and arches occurs and that the arches are loaded at five points.

Strains were measured in the same truss members for both tests. Tables 10 and 11 list the percent reduction (or increase) in truss live load member stresses between the initial and final tests at each load position. In order to facilitate this comparison between two tests conducted at different truck loads, the initial test stresses were multiplied by 1.507, the ratio of final to initial total truck weights. A zero percent reduction indicates no change in member stresses while a 100% reduction indicates a zero member stress during the final load test.

The design of the arch-superposition scheme changed the load application point on the interior vertical members. The load is now applied above the lower chord pin causing the member to act as a hanger rod. Therefore, a stress increase in the interior vertical members can be expected for certain load positions. Since the interior vertical

Table 10: Percent reduction (or increase) in truss live load member stresses - upstream truss

Member Number	Loading Position 1	2	3	4
1	43	65	80	94
5	95	85	65	39
6	10	56	18	41
7	63	49	55	(52)
8	89	56	55	100
9	--	--	--	--
10	29	30	33	33
11	78	--	--	--
12	17	(105)	81	80
13	85	70	(7)	13
14	0	0	86	90
15	--	44	51	68
16	9	20	(2)	30
17	(45)	(7)	(46)	33
18	(58)	0	34	38

Note: For member number designations see Figure 1. Percent reduction calculated using the absolute values of member stresses.

Table 11: Percent reduction (or increase) in truss live load member stresses - downstream truss

Member Number	Loading Position 1	2	3	4
1	(8)	3	52	48
5	80	46	40	0
6	53	50	29	52
7	66	98	72	0
8	96	62	47	65
9	(283)	(336)	(133)	22
10	40	71	48	(900)
11	82	--	100	0
12	97	0	53	94
13	65	55	(232)	48
14	61	50	78	83
15	--	--	59	58
16	(1185)	(160)	(83)	(144)
17	(94)	(128)	(26)	(7)
18	36	(1721)	(119)	56

Note: For member number designations see Figure 1. Percent reduction calculated using the absolute values of member stresses.

member is still connected to the lower chord chord pin, an increase in the counter tensile stresses can also be expected. Finally, because of the stress increase in the interior vertical members and the increased lateral stability provided by the arches and tension counter, the second counter is able to withstand a greater compressive live load (although the combined dead and live load stress is small). Although increased stresses in the counters can be expected, it should be noted that the arch system is designed to carry the entire H20 live load plus the combined arch and truss dead loads. Therefore, the increased stresses are not critical and do not affect the bridge capacity.

The member stresses in the downstream truss have been reduced an average of 56%. The member stresses in the upstream truss have been reduced an average of 53%.

Similarly, Table 12 lists the percent reduction (or increase) in truss deflections. Gauge 1 is located on the upstream truss, gauge 2 the downstream truss. The truss deflection is reduced an average of 41% for the upstream truss and 80% for the downstream truss. A greater reduction for the downstream truss is expected since the rehabilitation has reduced the effect of the loose lower chord joints of the downstream truss by bypassing them.

The average reduction in truss deflections of 41% and 80% for the upstream and downstream trusses of bridge number

Table 12: Percent reduction in truss deflections. Positive deflection downwards.

Gauge		Loading Position 1	2	3	4
	Extrapolated Truss Reading	.045	.130	.151	.056
1	Truss-Arch Reading	.001	.049	.084	-.078
	Percent Reduction	98	62	44	(39)
	Extrapolated Truss Reading	.048	.301	.342	.208
2	Truss-Arch Reading	-.001	.080	.123	.034
	Percent Reduction	98	73	64	84

50 straddles the average reduction of 69% found for the second bridge to have the arch-superposition scheme installed. The truss-arch model tested by Brungraber et al. [2] reduced truss deflections by an average of 30% to 40% with a maximum of 68%.

Clearly, the installation of the arch-superposition scheme has reduced the truss member stresses and deflections.

5. SUMMARY AND DISCUSSION

Chapter 2 described the original truss structure and the initial test results. The upstream truss has only one damaged member, and consequently the experimental and theoretical member stresses and truss deflections are in reasonable agreement. The downstream truss has five significantly damaged members and loose lower chord joints. The experimental and theoretical member stresses differed somewhat and the truss deflections were two times greater than expected.

A computer model of the bridge was found to provide a reasonable estimate of the member stresses. Therefore, it can be used to predict maximum stresses for a bridge rating.

A reasonable correlation between actual and predicted member stresses and truss deflections was found by Sanders et al. [5] and C. P. Heins [6] in tests of various through truss bridges. The predicted values were calculated by a determinate method of analysis assuming ideal pin connections.

Chapter 3 described the arch-superposition scheme and the final test results. The experimental results showed that sharing of the applied live load between the arches and trusses occurs. Also, the load distribution to the four arches is fairly even. The arch end supports are acting as 40% or 45% rotationally fixed bearings, due in part to the neoprene pads placed under the steel bearing plates which allow some movement.

Comparing the final test truss actual member stresses with the initial test stresses (multiplied by 1.507 to account for the difference in truck weights between the two tests), show an average reduction of 53% and 56% for the upstream and downstream trusses, respectivly. The truss deflections are reduced by an average of 41% (upstream) and 80% (downstream). The truss results also show no tied arch action occuring since the lower chord tensile stresses are not higher than predicted by the computer model.

A computer model was developed providing a reasonable correlation between the actual arch and truss stresses and the predicted values. It is recommended for design purposes when determining maximum member stresses, to assume a pin-pin arch support condition and both pin-pin and pin-roller truss support conditions. These maximum member stresses can then be used to design the arch section and check the adequacy of the truss members.

Located in Coudersport, Potter County, Pennsylvania, the second bridge to have the arch-superposition scheme installed was tested resulting in a 69% average reduction in truss deflections. The test of a model truss bridge by Brungraber et al. [2] resulted in an average reduction of 30% to 40% in truss deflections after the arches were installed. Also, to demonstrate the redundancy introduced by the arches, two tests were conducted on the model. In the first test, one truss lower chord member was removed (to simulate a failure). In the second test, one lower chord member was removed from both trusses. Although the bridge deflections increased, catastrophic failure was prevented by the arches.

The load testing and rehabilitation of Northumberland County bridge number 50 have demonstrated that the bridge capacity has been increased to H20-44 inventory and the scheme can be installed with just minimal traffic delays. This has been achieved at a considerably lower cost than would be incurred for a new structure built to today's specifications.

The total project cost, including all materials, fabrication and field work was $34,000. A replacement bridge would have to be 32 feet wide (two ten foot lanes plus six foot shoulders). For estimating purposes a cost of $110 per square foot is a reasonable value (1983) [9]. Therefore, it is estimated that a new bridge would cost $250,000, approximately seven times more than the arch-superposition scheme. Both the scheme and the new bridge result in 20 ton bridge load capacities.

For further details on estimating the cost of installing the arch-superposition scheme to truss bridges of varying length, see reference 9.

Several other methods to increase the load capacity of through truss bridges have been studied [8]. These are:

(1) stiffening of the top chord compression member

(2) prestressing of the lower chord with cables strung from the end supports

(3) shifting of the end supports

(4) adding a central support

Stiffening of the top chord is achieved by shortening the span length and thus reducing the slenderness ratio. For a Warren truss this is done by creating subpanels by adding the vertical members as shown in Figure 14.

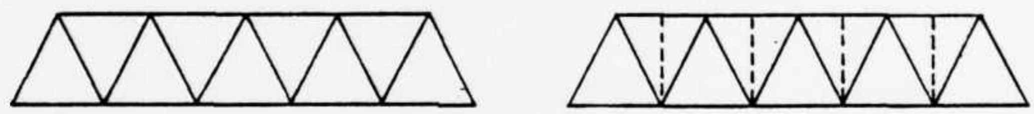


Figure 14: Stiffening of the top chord to increase bridge capacity.

Prestressing of the lower chord with a compressive force removes the dead load stress from the lower chord and results in lower live load tensile stresses. For a Warren truss prestressing is achieved by stringing a cable between the end supports and placing vertical struts between this cable and the lower chord as shown in Figure 15.

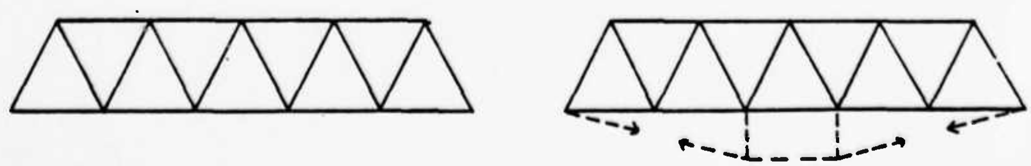


Figure 15: Prestressing of the lower chord to increase bridge capacity.

Shifting of the end supports inwards reduces the effective bridge span length and thus the member forces by as much as 40% to 50% (see Figure 16). Addition of a central support halves the span length while maintaining the span continuity.

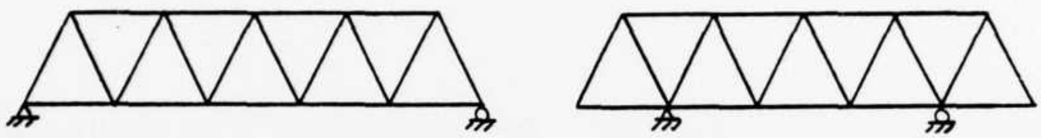


Figure 16: Shifting of the end supports to increase bridge capacity.

Although they are inexpensive when compared with the cost of a new bridge, there are several problems with the truss strengthening methods discussed so far. The determination of connection strengths remains critical. Even after completion of the work, maintenance of the truss members and connections is very important. Therefore, the difficulty in determining the bridge rating will remain. Finally, the improvement in load capacity is limited. With at most a 50% improvement in strength expected by any of the schemes, the truss must be rated at 13 tons prior to rehabilitation in order to achieve a 20 ton load capacity.

The arch-superposition scheme has been demonstrated to be effective in increasing the truss and roadway load capacities. Because of the redundancy provided by the arches, the risk of catastrophic failure is reduced.

6. CONCLUSIONS

Based upon the load testing and field work completed, the arch-superposition scheme applied to Northumberland County bridge number 50 was:

(1) structurally effective. The bridge load capacity was increased from H8-44 to H20-44.

(2) economically effective. The scheme cost a total of $34,000 as compared with $250,000 for a new bridge built to today's specifications.

(3) easily installed. Rehabilitation required 15 days using a two and three man crew.

(4) non-disruptive. The maximum traffic delay during the 15 day construction period was 15 minutes.

The results from tests conducted on the two truss-arch bridges and the model bridge indicate that a 50% average reduction in member stresses and truss deflections can be expected after the arch installation. Further, the arches provide a redundancy to the bridge as demonstrated in the model testing. The scheme is considered to be readily adaptable to truss bridges with various span lengths, heights, widths, and number of spans.

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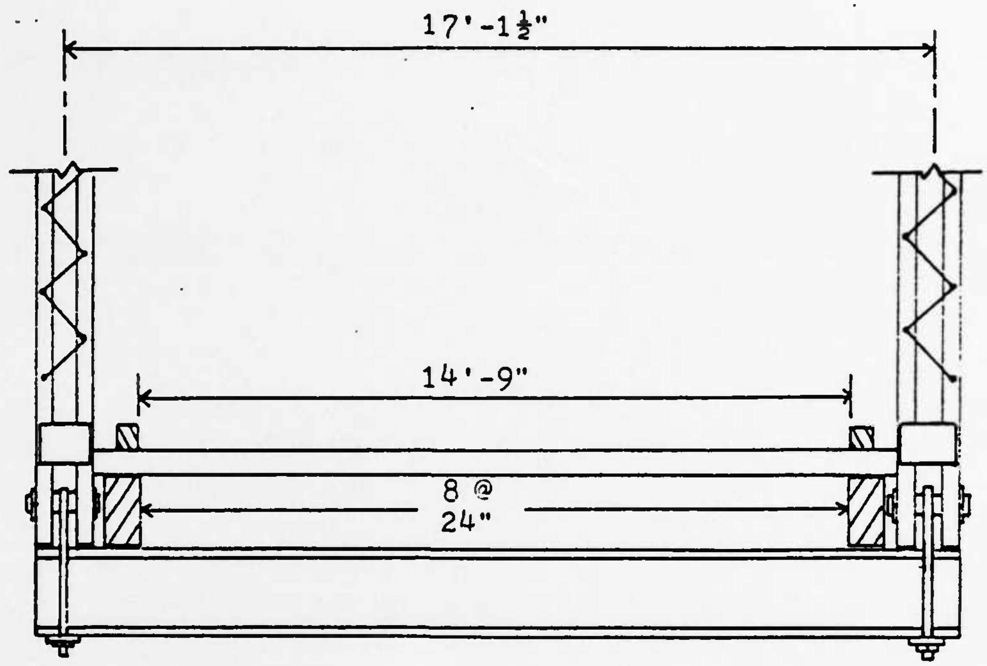
APPENDICES

Appendix 1: Bridge Rating Calculations

Nomenclature

b = length of member base (inches)
d = length of member height (inches)
FB = allowable bending strength (lbs/sq. inch)
fv = actual shear stress (lbs/sq. inch)
FV = allowable shear stress (lb/sq. inch)
FY = yield stress of the steel (lb/sq. inch)
L = span length (inches)
MD = dead load moment (lb-in)
ML = live load moment (lb-in)
MDL = dead plus live load moment (lb-in)
P = wheel load
S = average stringer spacing (feet)
SX = section modulus (cubic inches)
w = uniform load (lb/ft)

Note: Bridge rated by the allowable stress method from the "Manual for Maintenance Inspection of Bridges, 1978"



TIMBER DECK ANALYSIS

Use the following maximum allowable stresses:

FB=(1.33)x(1200)=1596   FV=120

For 2 x 4 inch planks: SX = (b)x(d)x(d)/6
= (1.5)x(3.5)x(3.5) = 3.06

MDL = (SX)x(FB) = (3.06)x(1596) = 4884

Timber Dead Load = (1.5)x(3.5)x(1)x(50)x(1/1728) = 0.15lb/in

MD = (w)x(l)x(l)/8 = (.15)x(24)x(24)/8 = 11.0

ML = 4884-11 = 4873

Distribution of Wheel Loads

Area of Contact:
Normal to Direction of Span = 15"
Direction of Span H10 = 14", H15 = 17", H20 = 20"
(based upon the equations: Area = .01P, L/W = 1/2.5)

By trial and error:

H15 $A = (17)x(15)$

$M = (P/A)x(1.5)x(17/2)x(24/2)-(P/A)x(1.5)x(8.5)x(4.25)$

$= 4873$

$(.39)xP = 4873$

$P = 12,575 > 12,000$

Check Shear

$fv = ((3)x(V))/((2)x(b)x(d))$

$V = ((12,000)/A)x(1.5)x(17/2) = 600$

$fv = ((3)x(600))/((2)x(1.5)x(3.5)) = 171 > 120 = FV$

Shear controls. The allowable load is less than 15 tons.

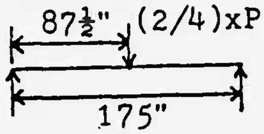
H10 $A = (14)x(15)$ $P = 8,000$

$V = ((8,000)/A)x(1.5)x(14/2) = 400$

$fv = ((3)x(400))/((2)x(1.5)x(3.5)) = 114 < 120$

TIMBER DECK IS RATED AT H10 OPERATING

STRINGER ANALYSIS



Use the following values:

L=175   S=2   FB=(1.33)x(1200)=1596   FV=120

Live load distribution to stringer = (2/4)x(P)

SX = (6)x(12)x(12)/6 = 144

MDL = (144)x(1596) = 229,824

Total dead load = 39.58 lb/ft

Deck: (3.5/12)x(1)x(50) = 14.58 lb/ft
Stringer: (6/12)x(1)x(50) = 25 lb/ft
(unit weight of timber = 50 lb/cu. ft)

MD = (39.58/12)x(175)x(175)/8 = 12,626

ML = (2/4)x(P)x(1)/4 = 229,824-12,626
P = 9,930

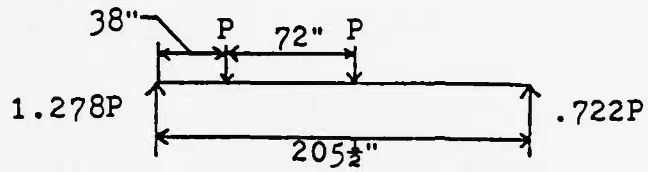
9,930/.8 = 12,411

Check shear for H12

V = (9600)x(2/4) = 4800
fv = ((3)x(4800))/((2)x(6)x(12)) = 100 < 120 O.K.

STRINGERS ARE RATED AT H12 OPERATING

FLOOR BEAM ANALYSIS



ML(max) = (.722xP)x(205.5-110.1)
= 68.85xP

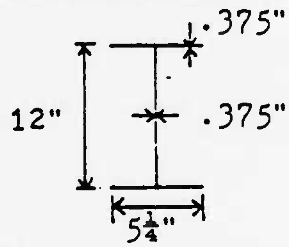
Use an impact factor of 1.3

Total ML(max) = (1.3)x(68.85xP) = 89.51xP

Total Dead Load on Floor Beam: 259.4 lb/ft

Deck: (3.5/12)x(14.58)x(50) = 212.6
8 Stringers: (6/12)x(14.58)x(50) = 21.3
Approximate Floor Beam: (8.15/144)x(450) = 25.5
(unit weight for cast iron is 450 lb/cu. ft)

MD = (21.6)x(205.5)x(205.5)/8 = 114,022



Existing floor beam dimensions as measured in the field.

Moment of Inertia = 177.6 IN^4

SX = 177.6/6 = 29.6

FB = (.75)x(FY) = (.75)x(26,000) = 19,500
(For operating rating only)

MDL = (19,500)x(29.6) = 577,200
(68.85xP)+(114,022) = 577,200
P = 6,727

6,727/.8 = 8,400

Check shear for H8 loading

FV = (.45)x(FY) = 11,700
fv = P/A = 6,400/((12)x(.375)) = 1,422 < 11,700 O.K.

FLOOR BEAMS ARE RATED FOR H8 OPERATING

FB = (.60)x(FY) = (.6)x(26,000) = 15,600
(For inventory rating)

(68.85xP)+(114,022) = 461,760
P = 5050

5050/.8 = 6,300

FLOOR BEAMS ARE RATED FOR H6 INVENTORY

Since the floor beams have the lowest operating rating among the deck, stringers, and floor beams, the bridge is rated at 8 tons operating and 6 tons inventory.

Appendix 2: Sample Computer Output

(1) Load Position 2 - Upstream truss with pin-pin supports

E for structure
30000.00
NJ
10

Jt	X	Y	Z	X-Coor	Y-Coor	X-Reaction	Y-Reaction	Z-Reaction
1	1	1	0	0.0000	0.0000	5.7444	5.9920	0.0000
2	0	0	0	175.00	192.00	0.0000	0.0000	0.0000
3	0	0	0	350.00	192.00	0.0000	0.0000	0.0000
4	0	0	0	525.00	192.00	0.0000	0.0000	0.0000
5	0	0	0	700.00	192.00	0.0000	0.0000	0.0000
6	1	1	0	875.00	0.0000	-5.7444	5.0280	0.0000
7	0	0	0	700.00	0.0000	0.0000	0.0000	0.0000
8	0	0	0	525.00	0.0000	0.0000	0.0000	0.0000
9	0	0	0	350.00	0.0000	0.0000	0.0000	0.0000
10	0	0	0	175.00	0.0000	0.0000	0.0000	0.0000

M
18

Mem	J	K	X-Area	Inertia	Length	COS	SIN
1	1	2	7.0200	25.660	259.79	.674	.739
2	2	3	7.0200	25.660	175.00	1.000	.000
3	3	4	7.0200	25.660	175.00	1.000	.000
4	4	5	7.0200	25.660	175.00	1.000	.000
5	5	6	7.0200	25.660	259.79	.674	-.739
6	7	6	1.9000	.10000E-03	175.00	1.000	.000
7	8	7	1.9000	.10000E-03	175.00	1.000	.000
8	9	8	2.8500	.10000E-03	175.00	1.000	.000
9	10	9	1.9000	.10000E-03	175.00	1.000	.000
10	1	10	1.9000	.10000E-03	175.00	1.000	.000
11	10	2	1.7600	.10000E-03	192.00	.000	1.000
12	9	3	3.0000	7.2900	192.00	.000	1.000
13	8	4	3.0000	7.2900	192.00	.000	1.000
14	7	5	1.7600	.10000E-03	192.00	.000	1.000
15	2	9	2.0000	.10000E-03	259.79	.674	-.739
16	3	8	1.0000	.10000E-03	259.79	.674	-.739
17	9	4	1.0000	.10000E-03	259.79	.674	.739
18	8	5	2.0000	.10000E-03	259.79	.674	.739

Truss with pin-pin supports - page 2

NLJ
2

Jt	HorLoad	VertLoad	RotLoad
8	0.0000	-3.1000	0.0000
9	0.0000	-7.9200	0.0000

NULM
0

Mem Uniform

Joint Displacements

Jt	Horizontal	Vertical	Rotational
1	0.0000	0.0000	-.12077E-04
2	.15930E-01	-.28063E-01	-.33011E-03
3	.38679E-02	-.39274E-01	-.17487E-03
4	-.14804E-02	-.76145E-01	.25596E-03
5	-.70893E-02	-.19649E-01	.26247E-03
6	0.0000	0.0000	-.16020E-04
7	.35397E-02	-.19649E-01	.22117E-03
8	.70794E-02	-.74070E-01	-.61095E-04
9	-.16768E-02	-.91376E-01	.20673E-04
10	-.83838E-03	-.28063E-01	-.25985E-03

Member-End Actions

MEM	J-Axial	J-Shear	J-Moment	K-Axial	K-Shear	K-Moment
1	8.1142	-.72552E-02	-.68549E-05	-8.1142	.72552E-02	-1.8848
2	10.906	.29345E-01	1.8848	-10.906	-.29345E-01	3.2506
3	10.046	-.10399E-01	-2.8051	-10.046	.10399E-01	.98529
4	9.1568	-.19189E-01	-1.7077	-9.1568	.19189E-01	-1.6505
5	6.6090	.63531E-02	1.6504	-6.6090	-.63531E-02	.37403E-05
6	1.1529	-.11409E-07	.30678E-05	-1.1529	.11409E-07	-.50644E-05
7	1.1529	-.27161E-06	-.28604E-04	-1.1529	.27161E-06	-.18927E-04
8	-4.2780	-.13987E-06	-.10837E-04	4.2780	.13987E-06	-.13640E-04

Truss with pin-pin supports - page 3

9	.27307	.28472E-06	.20104E-04	-.27307	-.28472E-06	.29722E-04
10	.27307	.28689E-07	.67575E-05	-.27307	-.28689E-07	-.17369E-05
11	-.51768E-05	-.20277E-06	-.18367E-04	.51768E-05	.20277E-06	-.20564E-04
12	-.98531	-.23205E-02	-.32168E-04	.98531	.23205E-02	-.44550
13	.96325	.37626E-02	.60377E-04	-.96325	-.37626E-02	.72236
14	-.11763E-05	.17192E-06	.15859E-04	.11763E-05	-.17192E-06	.17149E-04
15	-8.0673	.31762E-07	.74782E-07	3.0678	-.31762E-07	.81764E-05
16	1.2794	-.84257E-07	-.12258E-04	-1.2794	.84257E-07	-.96306E-05
17	-1.3152	.53013E-07	.41689E-05	1.3152	-.53013E-07	.96032E-05
18	-6.7772	-.46129E-07	-.97284E-05	6.7772	.46129E-07	-.22554E-05

Worst Case Stresses and Moment on Member

MEM	Axial	J-Shear	K-Shear	Maximum Moment
1	-1.1559	-.10335E-02	.10335E-02	1.8848
2	-1.5536	.41802E-02	-.41802E-02	3.2506
3	-1.4311	-.14813E-02	.14813E-02	2.8051
4	-1.3044	-.27335E-02	.27335E-02	1.7077
5	-.96994	.90499E-03	-.90499E-03	1.6504
6	-.60681	-.60047E-08	.60047E-08	.50644E-05
7	-.60681	-.14295E-06	.14295E-06	.28604E-04
8	1.5011	-.49077E-07	.49077E-07	.13640E-04
9	-.14372	.14985E-06	-.14985E-06	.29722E-04
10	-.14372	.15100E-07	-.15100E-07	.67575E-05
11	.29414E-05	-.11521E-06	.11521E-06	.20564E-04
12	.32844	-.77350E-03	.77350E-03	.44550
13	-.32108	.12542E-02	-.12542E-02	.72236
14	.66833E-06	.97680E-07	-.97680E-07	.17149E-04
15	4.0339	.15881E-07	-.15881E-07	.81764E-05
16	-1.2794	-.84257E-07	.84257E-07	.12258E-04
17	1.3152	.53013E-07	-.53013E-07	.96032E-05
18	3.3866	-.23065E-07	.23065E-07	.97284E-05

(2) Load Position 2 - Upstream truss with pin-roller supports

E for structure
30000.00
NJ
10

Jt	X	Y	Z	X-Coor	Y-Coor	X-Reaction	Y-Reaction	Z-Reaction
1	1	1	0	0.0000	0.0000	-.90837E-04	5.9920	0.0000
2	0	0	0	175.00	192.00	0.0000	0.0000	0.0000
3	0	0	0	350.00	192.00	0.0000	0.0000	0.0000
4	0	0	0	525.00	192.00	0.0000	0.0000	0.0000
5	0	0	0	700.00	192.00	0.0000	0.0000	0.0000
6	0	1	0	875.00	0.0000	0.0000	5.0280	0.0000
7	0	0	1	700.00	0.0000	0.0000	0.0000	-.66656E-04
8	0	0	0	525.00	0.0000	0.0000	0.0000	0.0000
9	0	0	0	350.00	0.0000	0.0000	0.0000	0.0000
10	0	0	1	175.00	0.0000	0.0000	0.0000	.74374E-04

M
18

Mem	J	K	X-Area	Inertia	Length	COS	SIN
1	1	2	7.0200	25.660	259.79	.674	.739
2	2	3	7.0200	25.660	175.00	1.000	.000
3	3	4	7.0200	25.660	175.00	1.000	.000
4	4	5	7.0200	25.660	175.00	1.000	.000
5	5	6	7.0200	25.660	259.79	.674	-.739
6	7	6	1.9000	.10000E-03	175.00	1.000	.000
7	3	7	1.9000	.10000E-03	175.00	1.000	.000
8	9	8	2.8500	.10000E-03	175.00	1.000	.000
9	10	9	1.9000	.10000E-03	175.00	1.000	.000
10	1	10	1.9000	.10000E-03	175.00	1.000	.000
11	10	2	1.7600	.10000E-03	192.00	.000	1.000
12	9	3	3.0000	7.2900	192.00	.000	1.000
13	8	4	3.0000	7.2900	192.00	.000	1.000
14	7	5	1.7600	.10000E-03	192.00	.000	1.000
15	2	9	2.0000	.10000E-03	259.79	.674	-.739
16	3	8	1.0000	.10000E-03	259.79	.674	-.739
17	9	4	1.0000	.10000E-03	259.79	.674	.739
18	8	5	2.0000	.10000E-03	259.79	.674	.739

NLJ
2

Jt	HorLoad	VertLoad	RotLoad
8	0.0000	-3.1000	0.0000
9	0.0000	-7.9200	0.0000

NULM
0

Mem Uniform

Joint Displacements

Jt	Horizontal	Vertical	Rotational
1	0.0000	0.0000	-.27997E-03
2	.56897E-01	-.65397E-01	-.43438E-03
3	.47834E-01	-.13246	-.17722E-03
4	.39275E-01	-.11935	.25832E-03
5	.31666E-01	-.56983E-01	.36674E-03
6	.31721E-01	0.0000	.25187E-03
7	.67640E-01	-.56983E-01	0.0000
8	.53559E-01	-.11669	-.17551E-04
9	.33565E-01	-.13397	-.22871E-04
10	.16783E-01	-.65397E-01	0.0000

Member-End Actions

MEM	J-Axial	J-Shear	J-Moment	K-Axial	K-Shear	K-Moment
1	3.1108	-.35225E-02	-.19994E-04	-3.1108	.35225E-02	-.91508
2	10.906	.23386E-01	.91509	-10.906	-.23386E-01	3.1775
3	10.300	-.10399E-01	-2.8258	-10.300	.10399E-01	1.0060
4	9.1572	-.13230E-01	-1.6346	-9.1572	.13230E-01	-.68075
5	6.8056	.26204E-02	.68073	-6.8056	-.26204E-02	.16267E-04
6	-4.5864	-.23472E-06	-.24856E-04	4.5864	.23472E-06	-.16220E-04
7	-4.5864	-.41137E-06	-.36296E-04	4.5864	.41137E-06	-.35694E-04
8	-9.7634	-.13987E-06	-.12330E-04	9.7634	.13987E-06	-.12147E-04

9	-5.4663	.44721E-06	.39523E-04	5.4663	-.44721E-06	.38739E-04
10	-5.4664	.27473E-06	.19239E-04	5.4664	-.27473E-06	.28838E-04
11	-.97623E-05	-.80672E-08	.60127E-05	.97623E-05	.80672E-08	-.75616E-05
12	-.70068	-.18318E-02	-.36590E-04	.70068	.18318E-02	-.35167
13	1.2479	.32739E-02	.64699E-04	-1.2479	-.32739E-02	.62853
14	-.86281E-05	-.39057E-08	-.61052E-05	.86281E-05	.39057E-08	.53553E-05
15	-8.0759	.83080E-08	-.36729E-05	8.0759	-.83080E-08	.58312E-05
16	.90238	-.82478E-07	-.12557E-04	-.90238	.82478E-07	-.88694E-05
17	-1.6922	.51234E-07	.34078E-05	1.6922	-.51234E-07	.99021E-05
18	-6.7853	-.22676E-07	-.73832E-05	6.7853	.22676E-07	.14922E-05

Worst Case Stresses and Moment on Member

MEM	Axial	J-Shear	K-Shear	Maximum Moment
1	-1.1554	-.50178E-03	.50178E-03	.91508
2	-1.5536	.33313E-02	-.33313E-02	3.1775
3	-1.4673	-.14813E-02	.14813E-02	2.8258
4	-1.3044	-.18847E-02	.18847E-02	1.6346
5	-.96946	.37328E-03	-.37328E-03	.68073
6	2.4139	-.12354E-06	.12354E-06	.24856E-04
7	2.4139	-.21651E-06	.21651E-06	.36296E-04
8	3.4275	-.49077E-07	.49077E-07	.12330E-04
9	2.8770	.23537E-06	-.23537E-06	.39523E-04
10	2.8770	.14459E-06	-.14459E-06	.28838E-04
11	.55468E-05	-.45836E-08	.45836E-08	.75616E-05
12	.23356	-.61060E-03	.61060E-03	.35167
13	-.41596	.10913E-02	-.10913E-02	.62853
14	.49023E-05	-.22191E-08	.22191E-08	.61052E-05
15	4.0380	.41540E-08	-.41540E-08	.58312E-05
16	-.90238	-.82478E-07	.82478E-07	.12557E-04
17	1.6922	.51234E-07	-.51234E-07	.99021E-05
18	3.3926	-.11338E-07	.11338E-07	.73832E-05

(3) Load Position 2 - Upstream truss-arch with pin-pin arch supports

E for structure
30000.00
NJ
30

Jt	X	Y	Z	X-Coor	Y-Coor	X-Reaction	Y-Reaction	Z-Reaction
1	1	1	0	0.0000	3.2000	5.2359	5.8667	0.0000
2	0	0	0	174.96	195.20	0.0000	0.0000	0.0000
3	0	0	0	349.92	195.20	0.0000	0.0000	0.0000
4	0	0	0	524.88	195.20	0.0000	0.0000	0.0000
5	0	0	0	699.84	195.20	0.0000	0.0000	0.0000
6	1	1	0	874.80	3.2000	-5.3509	4.2971	0.0000
7	1	1	0	847.80	0.0000	-4.8401	2.7846	0.0000
8	0	0	1	787.32	3.2000	0.0000	0.0000	.52322E-03
9	0	0	1	699.84	3.2000	0.0000	0.0000	.45858E-04
10	0	0	1	612.36	3.2000	0.0000	0.0000	-.32219E-03
11	0	0	0	524.88	3.2000	0.0000	0.0000	0.0000
12	0	0	1	437.40	3.2000	0.0000	0.0000	-.63532E-04
13	0	0	0	349.92	3.2000	0.0000	0.0000	0.0000
14	0	0	1	262.44	3.2000	0.0000	0.0000	.23980E-03
15	0	0	1	174.96	3.2000	0.0000	0.0000	.85290E-04
16	0	0	1	87.480	3.2000	0.0000	0.0000	-.23467E-03
17	1	1	0	27.000	0.0000	4.9529	3.0822	0.0000
18	0	0	0	87.480	47.915	0.0000	0.0000	0.0000
19	0	0	0	174.96	103.73	0.0000	0.0000	0.0000
20	0	0	0	262.44	143.60	0.0000	0.0000	0.0000
21	0	0	0	349.92	167.53	0.0000	0.0000	0.0000
22	0	0	0	437.40	175.50	0.0000	0.0000	0.0000
23	0	0	0	524.88	167.53	0.0000	0.0000	0.0000
24	0	0	0	612.36	143.60	0.0000	0.0000	0.0000
25	0	0	0	699.84	103.73	0.0000	0.0000	0.0000
26	0	0	0	787.32	47.915	0.0000	0.0000	0.0000
27	0	0	0	174.96	15.200	0.0000	0.0000	0.0000
28	0	0	0	349.92	15.200	0.0000	0.0000	0.0000
29	0	0	0	524.88	15.200	0.0000	0.0000	0.0000
30	0	0	0	699.34	15.200	0.0000	0.0000	0.0000

Truss-arch with pin-pin arch supports - page 2

M
46

Mem	J	K	X-Area	Inertia	Length	COS	SIN
1	1	2	7.0200	25.660	259.76	.674	.739
2	2	3	7.0200	25.660	174.96	1.000	.000
3	3	4	7.0200	25.660	174.96	1.000	.000
4	4	5	7.0200	25.660	174.96	1.000	.000
5	5	6	7.0200	25.660	259.76	.674	-.739
6	8	6	1.9000	.10000E-03	87.480	1.000	.000
7	9	8	1.9000	.10000E-03	87.480	1.000	.000
8	10	9	1.9000	.10000E-03	87.480	1.000	.000
9	11	10	1.9000	.10000E-03	87.480	1.000	.000
10	12	11	2.8500	.10000E-03	87.480	1.000	.000
11	13	12	2.8500	.10000E-03	87.480	1.000	.000
12	14	13	1.9000	.10000E-03	87.480	1.000	.000
13	15	14	1.9000	.10000E-03	87.480	1.000	.000
14	16	15	1.9000	.10000E-03	87.480	1.000	.000
15	1	16	1.9000	.10000E-03	87.480	1.000	.000
16	17	18	12.180	258.00	77.160	.784	.621
17	18	19	12.180	258.00	103.77	.843	.538
18	19	20	12.180	258.00	96.137	.910	.415
19	20	21	12.180	258.00	90.694	.965	.264
20	21	22	12.180	258.00	87.842	.996	.091
21	22	23	12.180	258.00	87.842	.996	-.091
22	23	24	12.180	258.00	90.694	.965	-.264
23	24	25	12.180	258.00	96.137	.910	-.415
24	25	26	12.180	258.00	103.77	.843	-.538
25	26	7	12.180	258.00	77.160	.784	-.621
26	16	18	1.6700	.10000E-03	44.715	.000	1.000

Truss-arch with pin-pin arch supports - page 3

```
27  27 19   4.4100       .10000E-03  88.530   .000  1.000
28  19  2   .10000E-03   .10000E-03  91.470   .000  1.000
29  14 20   1.6700       .10000E-03  140.40   .000  1.000
30  28 21   3.0000       7.2900      152.33   .000  1.000
31  21  3   3.0000       .10000E-03  27.670   .000  1.000
32  12 22   1.6700       .10000E-03  172.30   .000  1.000
33  29 23   3.0000       7.2900      152.33   .000  1.000
34  23  4   3.0000       7.2900      27.670   .000  1.000
35  10 24   1.6700       .10000E-03  140.40   .000  1.000
36  30 25   4.4100       .10000E-03  88.530   .000  1.000
37  25  5   .10000E-03   .10000E-03  91.470   .000  1.000
38   8 26   1.6700       .10000E-03  44.715   .000  1.000
39   2 13   2.0000       .10000E-03  259.76   .674  -.739
40   3 11   1.6700       .10000E-03  259.76   .674  -.739
41  13  4   1.6700       .10000E-03  259.76   .674   .739
42  11  5   2.0000       .10000E-03  259.76   .674   .739
43  15 27   4.4100       .10000E-03  12.000   .000  1.000
44  13 28   3.0000       7.2900      12.000   .000  1.000
45  11 29   3.0000       7.2900      12.000   .000  1.000
46   9 30   4.4100       .10000E-03  12.000   .000  1.000
NLJ
  5
 Jt  HorLoad     VertLoad    RotLoad
 10  0.0000      -.32000     0.0000



 12  0.0000      -2.4300     0.0000
 14  0.0000      -1.7800     0.0000
 28  0.0000      -8.9100     0.0000
 29  0.0000      -2.5900     0.0000
NULM
  0
```

Truss-arch with pin-pin arch supports - page 4

Mem Uniform

Joint Displacements

Jt	Horizontal	Vertical	Rotational
1	0.0000	0.0000	-.21153E-04
2	.14861E-01	-.26796E-01	-.29298E-03
3	.59868E-02	-.84154E-01	-.21447E-03
4	-.17066E-02	-.67949E-01	.44524E-03
5	-.82224E-02	-.17199E-01	.16400E-03
6	0.0000	0.0000	.19978E-04
7	0.0000	0.0000	-.11160E-02
8	.21957E-02	.60391E-01	0.0000
9	.43915E-02	.87061E-01	0.0000
10	.65872E-02	.25725E-01	0.0000
11	.87829E-02	-.66808E-01	-.54033E-03
12	.47551E-02	-.12188	0.0000
13	.72734E-03	-.86000E-01	.15438E-03
14	.54550E-03	-.36097E-01	0.0000
15	.36366E-03	.23984E-01	0.0000
16	.18182E-03	.28513E-01	0.0000
17	0.0000	0.0000	.56860E-03
18	-.24151E-01	.28513E-01	.31486E-03
19	-.23226E-01	.23984E-01	-.41326E-03
20	.21059E-03	-.31109E-01	-.68273E-03
21	.13536E-01	-.84638E-01	-.54267E-03
22	.14942E-01	-.11352	.54663E-04
23	.17927E-01	-.67437E-01	.91394E-03
24	.42288E-01	.26622E-01	.10340E-02
25	.68227E-01	.87061E-01	.22947E-03
26	.49332E-01	.60391E-01	-.79949E-03
27	-.10858E-02	.23984E-01	.22899E-03
28	-.11104E-02	-.87002E-01	.15067E-03
29	.15236E-01	-.67174E-01	-.53260E-03
30	.71925E-02	.87061E-01	-.44548E-03

Truss-arch with pin-pin arch supports - page 5

Member-End Actions

MEM	J-Axial	J-Shear	J-Moment	K-Axial	K-Shear	K-Moment
1	7.9427	-.62019E-02	.69827E-04	-7.9427	.62019E-02	-1.6111
2	10.681	.22365E-01	1.6110	-10.681	-.22365E-01	2.3020
3	9.2606	.68707E-02	-2.3016	-9.2606	-.68707E-02	3.5037
4	7.8430	.43911E-02	1.6216	-7.8430	-.43911E-02	-.85331
5	5.8166	.32850E-02	.85345	-5.8166	-.32850E-02	-.14659E-03
6	1.4307	.32945E-05	.14342E-03	-1.4307	-.32945E-05	.14479E-03
7	1.4307	.14342E-05	.62731E-04	-1.4307	-.14342E-05	.62731E-04
8	1.4307	-.32983E-05	-.14427E-03	-1.4307	.32983E-05	-.14427E-03
9	1.4307	-.62469E-05	-.29177E-03	-1.4307	.62469E-05	-.25471E-03
10	-3.9366	-.42325E-05	-.16660E-03	3.9366	.42325E-05	-.20366E-03
11	-3.9366	.22926E-05	.10557E-03	3.9366	-.22926E-05	.94986E-04
12	-.11848	.30466E-05	.12796E-03	.11848	-.30466E-05	.13855E-03
13	-.11848	.32308E-05	.14132E-03	.11848	-.32308E-05	.14132E-03
14	-.11848	.24358E-06	.10654E-04	.11848	-.24358E-06	.10654E-04
15	-.11847	-.15830E-05	-.69968E-04	.11847	.15830E-05	-.68517E-04
16	5.7962	-.65974	-.37386E-04	-5.7962	.65974	-50.905
17	5.8332	-.65608E-01	50.905	-5.8332	.65608E-01	-57.713
18	5.7841	.74930	57.713	-5.7841	-.74930	14.322
19	5.1202	-.52244E-01	-14.323	-5.1202	.52244E-01	9.5842
20	5.0776	1.0224	-7.7292	-5.0776	-1.0224	97.536
21	5.0300	-.49689	-97.537	-5.0300	.49689	53.889
22	5.3163	-1.0981	-60.045	-5.3163	1.0981	-39.550
23	5.5565	-.52474	39.550	-5.5565	.52474	-89.997
24	5.5775	.25534	89.997	-5.5775	-.25534	-63.501
25	5.5230	.82297	63.500	-5.5230	-.82297	-.27585E-05
26	.16026E-04	-.69634E-05	-.17681E-03	-.16026E-04	.69634E-05	-.13456E-03
27	.46570E-04	-.15719E-05	-.47817E-04	-.46570E-04	.15719E-05	-.91344E-04
28	.16655E-02	.27221E-06	.85047E-05	-.16655E-02	-.27221E-06	.16394E-04

29	-1.7800	-.62779E-06	-.29482E-04	1.7800	.62779E-06	-.58659E-04
30	-1.3963	-.11293E-01	.13530	1.3963	.11293E-01	-1.8556
31	-1.5748	-.30629E-04	-.45933E-03	1.5748	.30629E-04	-.38816E-03
32	-2.4300	.10484E-06	.80800E-05	2.4300	-.10484E-06	.99836E-05
33	.15523	.23562E-01	-.28221	-.15523	-.23562E-01	3.8714
34	1.6659	-.10269	2.2837	-1.6659	.10269	-5.1253
35	-.32000	.14086E-05	.76789E-04	.32000	-.14086E-05	.12098E-03
36	.26007E-03	.26706E-05	.95344E-04	-.26007E-03	-.26706E-05	.14109E-03
37	.34195E-02	-.27497E-05	-.12361E-03	-.34195E-02	.27497E-05	-.12790E-03
38	.23085E-05	.11783E-04	.31707E-03	-.23085E-05	-.11783E-04	.20979E-03
39	-7.9091	.66386E-07	.34555E-05	7.9091	-.66386E-07	.13789E-04
40	2.1096	-.22959E-06	-.26056E-04	-2.1096	.22959E-06	-.33583E-04
41	-2.2571	.13129E-06	.13693E-04	2.2571	-.13129E-06	.20411E-04
42	-5.8242	-.19484E-06	-.33440E-04	5.8242	.19484E-06	-.17171E-04
43	.95693E-05	-.15719E-05	-.66680E-04	-.95693E-05	.15719E-05	.47817E-04
44	7.5136	-.11299E-01	-.25675E-03	-7.5136	.11299E-01	-.13533
45	2.7452	.23556E-01	.52624E-03	-2.7452	-.23556E-01	.28215
46	.13499E-05	.26710E-05	.12740E-03	-.13499E-05	-.26710E-05	-.95344E-04

Worst Case Stresses and Moment on Member

MEM	Axial	J-Shear	K-Shear	Maximum Moment
1	-1.1314	-.88346E-03	.88346E-03	1.6111
2	-1.5216	.31859E-02	-.31859E-02	2.3020
3	-1.3192	.97873E-03	-.97873E-03	3.5037
4	-1.1172	.62551E-03	-.62551E-03	1.6216
5	-.82858	.46795E-03	-.46795E-03	.85345
6	-.75300	.17339E-05	-.17339E-05	.14479E-03
7	-.75299	.75483E-06	-.75483E-06	.62731E-04
8	-.75299	-.17360E-05	.17360E-05	.14427E-03
9	-.75299	-.32878E-05	.32878E-05	.29177E-03
10	1.3813	-.14851E-05	.14851E-05	.20366E-03

Truss-arch with pin-pin arch supports - page 7

11	1.3813	.80443E-06	-.80443E-06	.10557E-03
12	.62359E-01	.16035E-05	-.16035E-05	.13855E-03
13	.62359E-01	.17004E-05	-.17004E-05	.14132E-03
14	.62358E-01	.12820E-06	-.12820E-06	.10654E-04
15	.62354E-01	-.83318E-06	.83318E-06	.69968E-04
16	-.47588	-.54165E-01	.54165E-01	50.905
17	-.47892	-.53865E-02	.53865E-02	57.713
18	-.47489	.61519E-01	-.61519E-01	57.713
19	-.42038	-.42893E-02	.42893E-02	14.323
20	-.41688	.83938E-01	-.83938E-01	97.536
21	-.41297	-.40795E-01	.40795E-01	97.537
22	-.43648	-.90160E-01	.90160E-01	60.045
23	-.45620	-.43082E-01	.43082E-01	89.997
24	-.45792	.20964E-01	-.20964E-01	89.997
25	-.45344	.67567E-01	-.67567E-01	63.500
26	-.95962E-05	-.41697E-05	.41697E-05	.17681E-03
27	-.10560E-04	-.35644E-06	.35644E-06	.91344E-04
28	-16.655	.27221E-02	-.27221E-02	.16394E-04
29	1.0659	-.37592E-06	.37592E-06	.58659E-04
30	.46545	-.37643E-02	.37643E-02	1.8556
31	.52492	-.10210E-04	.10210E-04	.45933E-03
32	1.4551	.62777E-07	-.62777E-07	.99836E-05
33	-.51744E-01	.78540E-02	-.78540E-02	3.8714
34	-.55529	-.34231E-01	.34231E-01	5.1253
35	.19162	.84347E-06	-.84347E-06	.12098E-03
36	-.58974E-04	.60559E-06	-.60559E-06	.14109E-03
37	-34.195	-.27497E-01	.27497E-01	.12790E-03
38	-.13824E-05	.70555E-05	-.70555E-05	.31707E-03
39	3.9545	.33193E-07	-.33193E-07	.13789E-04
40	-1.2632	-.13748E-06	.13748E-06	.33583E-04
41	1.3516	.78617E-07	-.78617E-07	.20411E-04
42	2.9121	-.97420E-07	.97420E-07	.33440E-04
43	-.21699E-05	-.35645E-06	.35645E-06	.66680E-04
44	-2.5045	-.37664E-02	.37664E-02	.13533
45	-.91508	.78520E-02	-.78520E-02	.28215
46	-.30610E-06	.60566E-06	-.60566E-06	.12740E-03

(4) Load Position 2 - Upstream truss-arch with fixed-fixed arch supports

NLJ
5

Jt	HorLoad	VertLoad	RotLoad
10	0.0000	-.32000	0.0000
12	0.0000	-2.4300	0.0000
14	0.0000	-1.7800	0.0000
28	0.0000	-8.9100	0.0000
29	0.0000	-2.5900	0.0000

NULM
0
Mem Uniform

Joint Displacements

Jt	Horizontal	Vertical	Rotational
1	0.0000	0.0000	-.15323E-04
2	.12705E-01	-.23276E-01	-.25886E-03
3	.48741E-02	-.71994E-01	-.15360E-03
4	-.19069E-02	-.56507E-01	.30349E-03
5	-.72984E-02	-.14690E-01	.15714E-03
6	0.0000	0.0000	.97135E-05
7	0.0000	0.0000	0.0000
8	.18844E-02	.23494E-01	0.0000
9	.37688E-02	.59395E-01	0.0000
10	.56531E-02	.22700E-01	0.0000
11	.75375E-02	-.56406E-01	-.44085E-03
12	.41627E-02	-.10519	0.0000
13	.78788E-03	-.75006E-01	.16525E-03
14	.59091E-03	-.27856E-01	0.0000
15	.39394E-03	.19697E-01	0.0000
16	.19697E-03	.12941E-01	0.0000
17	0.0000	0.0000	0.0000

Truss-arch with fixed-fixed arch supports - page 2

18	-.12327E-01	.12941E-01	.30416E-03
19	-.19261E-01	.19697E-01	-.22328E-03
20	-.21085E-02	-.22868E-01	-.59968E-03
21	.96228E-02	-.72428E-01	-.49186E-03
22	.10178E-01	-.96828E-01	.64917E-04
23	.12240E-01	-.55987E-01	.79603E-03
24	.32078E-01	.23596E-01	.79744E-03
25	.46156E-01	.59395E-01	-.55489E-04
26	.20656E-01	.23494E-01	-.60763E-03
27	-.66104E-03	.19697E-01	.16708E-03
28	-.11811E-02	-.75919E-01	.16175E-03
29	.12801E-01	-.56696E-01	-.43428E-03
30	.53665E-02	.59395E-01	-.25506E-03

Member-End Actions

MEM	J-Axial	J-Shear	J-Moment	K-Axial	K-Shear	K-Moment
1	7.0105	-.55567E-02	.32457E-04	-7.0105	.55567E-02	-1.4434
2	9.4259	.21794E-01	1.4434	-9.4259	-.21794E-01	2.3697
3	8.1622	-.40953E-02	-2.3694	-8.1622	.40953E-02	1.6529
4	6.4898	-.26253E-02	.41428	-6.4898	.26253E-02	-.87360
5	4.8175	.33633E-02	.87371	-4.8175	-.33633E-02	-.57854E-04
6	1.2278	.12862E-05	.55927E-04	-1.2278	-.12862E-05	.56593E-04
7	1.2278	.19305E-05	.84441E-04	-1.2278	-.19305E-05	.84441E-04
8	1.2278	-.19733E-05	-.86310E-04	-1.2278	.19733E-05	-.86310E-04
9	1.2278	-.52908E-05	-.24654E-03	-1.2278	.52908E-05	-.21630E-03
10	-3.2984	-.36600E-05	-.14497E-03	3.2984	.36600E-05	-.17521E-03
11	-3.2984	.20116E-05	.93653E-04	3.2984	-.20116E-05	.82319E-04
12	-.12834	.29241E-05	.12223E-03	.12834	-.29241E-05	.13357E-03
13	-.12834	.25572E-05	.11185E-03	.12834	-.25572E-05	.11185E-03
14	-.12834	-.36335E-06	-.15893E-04	.12834	.36335E-06	-.15893E-04
15	-.12834	-.73191E-06	-.32539E-04	.12834	.73191E-06	-.31488E-04

Truss-arch with fixed-fixed arch supports - page 3

16	7.7021	-1.2260	-77.808	-7.7021	1.2260	-16.787
17	7.7869	-.43470	16.787	-7.7869	.43470	-61.895
18	7.7704	.65721	61.895	-7.7704	-.65721	1.2874
19	7.0956	.17452	-1.2876	-7.0956	-.17452	17.115
20	6.9069	.76710	-15.367	-6.9069	-.76710	82.751
21	6.8752	-.41737	-82.752	-6.8752	.41737	46.089
22	7.5070	-1.2125	-55.101	-7.5070	1.2125	-54.860
23	7.7372	-.28728	54.860	-7.7372	.28728	-82.479
24	7.7030	.79591	82.479	-7.7030	-.79591	.11182
25	7.5819	1.5770	-.11189	-7.5819	-1.5770	121.79
26	.35606E-05	-.23048E-05	-.71936E-04	-.35606E-05	.23048E-05	-.31123E-04
27	.42626E-04	-.10941E-05	-.35204E-04	-.42626E-04	.10941E-05	-.61660E-04
28	.14094E-02	.46641E-06	.22499E-04	-.14094E-02	-.46641E-06	.20164E-04
29	-1.7800	-.58270E-06	-.28092E-04	1.7800	.58270E-06	-.53719E-04
30	-2.0624	-.10646E-01	.12755	2.0624	.10646E-01	-1.7492
31	-1.4126	-.23244E-04	-.35826E-03	1.4126	.23244E-04	-.28491E-03
32	-2.4300	.81699E-07	.59081E-05	2.4300	-.81699E-07	.81687E-05
33	-.41874	.20040E-01	-.24002	.41874	-.20040E-01	3.2927
34	1.6920	.13197	5.7188	-1.6920	-.13197	-2.0672
35	-.32000	.10719E-05	.58208E-04	.32000	-.10719E-05	.92287E-04
36	.16898E-03	.14031E-05	.55346E-04	-.16898E-03	-.14031E-05	.68871E-04
37	.24298E-02	-.22958E-05	-.11197E-03	-.24298E-02	.22958E-05	-.98026E-04
38	.19628E-05	.20883E-05	.87457E-04	-.19628E-05	-.20883E-05	.59231E-05
39	-6.9779	.64683E-07	.35029E-05	6.9779	-.64683E-07	.13299E-04
40	1.8762	-.18419E-06	-.20605E-04	-1.8762	.18419E-06	-.27240E-04
41	-2.2871	.95360E-07	.10789E-04	2.2871	-.95360E-07	.13982E-04
42	-4.8141	-.15592E-06	-.27157E-04	4.8141	.15592E-06	-.13345E-04
43	-.50841E-06	-.10942E-05	-.48334E-04	.50841E-06	.10942E-05	.35204E-04
44	6.8475	-.10651E-01	-.22854E-03	-6.8475	.10651E-01	-.12758
45	2.1714	.20034E-01	.44325E-03	-2.1714	-.20034E-01	.23996
46	.72645E-04	.14033E-05	.72185E-04	-.72645E-04	-.14033E-05	-.55345E-04

Truss-arch with fixed-fixed arch supports - page 5

28	-14.094	.46641E-02	-.46641E-02	.22499E-04
29	1.0659	-.34892E-06	.34892E-06	.53719E-04
30	.68747	-.35486E-02	.35486E-02	1.7492
31	.47087	-.77481E-05	.77481E-05	.35826E-03
32	1.4551	.48922E-07	-.48922E-07	.81687E-05
33	.13958	.66800E-02	-.66800E-02	3.2927
34	-.56399	.43990E-01	-.43990E-01	5.7188
35	.19162	.64186E-06	-.64186E-06	.92287E-04
36	-.38318E-04	.31816E-06	-.31816E-06	.68871E-04
37	-24.298	-.22958E-01	.22958E-01	.11197E-03
38	-.11753E-05	.12505E-05	-.12505E-05	.87457E-04
39	3.4889	.32341E-07	-.32341E-07	.13299E-04
40	-1.1234	-.11029E-06	.11029E-06	.27240E-04
41	1.3695	.57102E-07	-.57102E-07	.13982E-04
42	2.4071	-.77960E-07	.77960E-07	.27157E-04
43	.11529E-06	-.24811E-06	.24811E-06	.48334E-04
44	-2.2825	-.35502E-02	.35502E-02	.12758
45	-.72379	.66779E-02	-.66779E-02	.23996
46	-.16473E-04	.31821E-06	-.31821E-06	.72185E-04

Truss-arch with fixed-fixed arch supports - page 4

Worst Case Stresses and Moment on Member

MEM	Axial	J-Shear	K-Shear	Maximum Moment
1	-.99865	-.79155E-03	.79155E-03	1.4434
2	-1.3427	.31046E-02	-.31046E-02	2.3697
3	-1.1627	-.58338E-03	.58338E-03	2.3694
4	-.92448	-.37397E-03	.37397E-03	.87360
5	-.68625	.47911E-03	-.47911E-03	.87371
6	-.64622	.67697E-06	-.67697E-06	.56593E-04
7	-.64622	.10161E-05	-.10161E-05	.84441E-04
8	-.64622	-.10386E-05	.10386E-05	.86310E-04
9	-.64622	-.27846E-05	.27846E-05	.24654E-03
10	1.1573	-.12842E-05	.12842E-05	.17521E-03
11	1.1573	.70581E-06	-.70581E-06	.93653E-04
12	.67549E-01	.15390E-05	-.15390E-05	.13357E-03
13	.67548E-01	.13459E-05	-.13459E-05	.11185E-03
14	.67548E-01	-.19123E-06	.19123E-06	.15893E-04
15	.67547E-01	-.38522E-06	.38522E-06	.32539E-04
16	-.63236	-.10065	.10065	77.808
17	-.63932	-.35689E-01	.35689E-01	61.895
18	-.63796	.53958E-01	-.53958E-01	61.895
19	-.58256	.14328E-01	-.14328E-01	17.115
20	-.56707	.62980E-01	-.62980E-01	82.751
21	-.56446	-.34267E-01	.34267E-01	82.752
22	-.61634	-.99544E-01	.99544E-01	55.101
23	-.63524	-.23586E-01	.23586E-01	82.479
24	-.63243	.65345E-01	-.65345E-01	82.479
25	-.62249	.12947	-.12947	121.79
26	-.21321E-05	-.13801E-05	.13801E-05	.71936E-04
27	-.96657E-05	-.24810E-06	.24810E-06	.61660E-04

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